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Featuring

Wind Bracing in Industrial and Many-Storied Buildings

Design of Details in Timber Structures

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JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

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WIND BRACING IN INDUSTRIAL AND MANY-STORIED BUILDINGS

By Robins Fleming*

(Presented before the Designers Section, Boston Society of Civil Engineers, on March 10, 1926)

[The text and figures in this paper are largely taken from contributions by the same author to technical periodicals, — mainly, the "Engineering News-Record", "Engineering" and the "Canadian Engineer".]

Wind is air in motion. It is a gravitational phenomenon induced and maintained by temperature differences. These differences arise from the unequal heating of the earth's surface by the sun, about three times as much heat being received at the equator as at the poles. Air of heavier weight seeks to replace that of lighter weight. Let us consider a weather map.

We might infer that wind currents are at right angles to the isobars (*i.e.*, lines of equal barometric pressure) and flow directly from those of high pressure to those of low pressure. This it would do were it not for the rotation of the earth. Because of this rotation the laws governing the circulation of the air are complicated and complex. The force which produces and maintains motion of the air is the gravitational pressure gradient. "Gradient winds" and "gradient velocity" are favorite terms with the meteorologist but they can only be alluded to here.

What the structural engineer is seeking to know, is the force of the wind. How is this force determined? Usually, but not always, by measuring its velocity. Attempts to do this have been made during the past two centuries by all manner of means. The Robinson cup anemometer, or some modification of it, is more used at present for this purpose than any other method. This instrument was brought out by Rev. Thomas R. Robinson, D.D., a member of the Royal Irish Academy, in 1850.

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* Structural Engineer, American Bridge Company, New York, N. Y.

A certain number of revolutions per unit of time represents the velocity of the wind. Dr. Robinson thought that the hemispherical cups moved with one-third the wind's velocity. But this has been disproved — the actual velocity is less than the indicated velocity. To obtain the actual velocity the United States Weather Bureau therefore applies a logarithmic formula of correction to the indicated or recorded velocity. It is the observed or indicated velocities that are published. The difference between indicated and actual velocities is shown in Fig. 1.

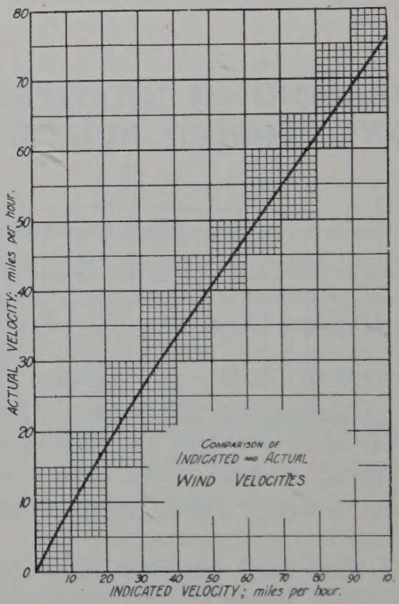


FIG. 1. — WIND VELOCITIES

Comparison of indicated and actual velocities

RELATION BETWEEN WIND VELOCITY AND PRESSURE

We come now to the relationship between velocity and pressure. The most careful experiments of recent years for the pressure on flat plates of moderate size, normal to the direction of the wind, give a value of P of from $0.0032V^2$ to $0.004V^2$ in which P is the pressure in pounds per square foot and V is the velocity in miles per hour. The United States Weather Bureau gives $P = 0.004V^2$, which may be rather high. Fig. 2 shows the difference in pressure between indicated and actual wind velocities.

An interesting question arises: Shall the indicated or actual pressure be used in assuming pressure to withstand given velocities? The recorded velocities are those sustained for a period of five minutes, but during that period there may be gusts of a minute or less of greater velocity. On the other hand, maximum wind pressures are less per unit of area on large surfaces than on small ones. The hemispherical cups of the Robinson anemometers are 4-in. diameter and the area of the side of a building may be thousands of square feet. Again, wind velocity increases with distance from the ground. Again, and most important of all, is, what is the maximum velocity that should be considered? The maximum recorded velocities of a number of cities are:

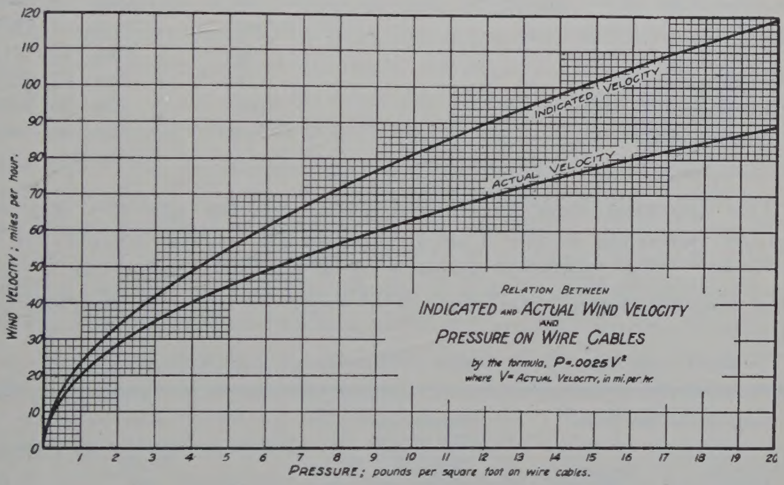


FIG. 2. — WIND VELOCITY AND PRESSURE

Showing difference in pressure between indicated and actual velocities

WIND VELOCITIES

[Reported by United States Weather Bureau]

STATION	Maximum Velocity for a Period of Five Minutes (Miles per Hour)	Date
Albany, N. Y.	70	Oct. 23, 1878
Boston, Mass.	72	March 9, 1877
Buffalo, N. Y.	96	Dec. 23, 1920
Chicago, Ill.	84	Feb. 12, 1894
Los Angeles, Cal.	48	Jan. 8, 1882
Miami, Fla.	46	Sept. 9, 1919
New York, N. Y.	96	Feb. 22, 1912
Philadelphia, Pa.	75	Oct. 23, 1878
St. Paul, Minn.	102	Aug. 20, 1904
San Francisco, Cal.	64	Nov. 30, 1906

The following table is from a paper by the eminent engineer, Smeaton, published in the Transactions of the Royal Society for the year 1759. The Smeaton formula, $P = 0.005V^2$, based on a very slender and erroneous foundation, held sway for one hundred and fifty years.

DATA CONCERNING THE WINDS, PREPARED BY SMEATON AND ROUSE

VELOCITY OF THE WIND		Perpendicular Force on One-foot Area in Pounds Avoirdupois	Common Appellation of the Force of Winds
Miles in One Hour	Feet in One Second		
1	1.47	0.005	Hardly perceptible.
2	2.93	0.020	Just perceptible.
3	4.40	0.044	
4	5.87	0.079	Gentle, pleasant wind.
5	7.33	0.123	
10	14.67	0.492	Pleasant, brisk gale.
15	22.00	1.107	
20	29.34	1.968	Very brisk.
25	36.67	3.075	
30	44.01	4.429	High winds.
35	51.34	6.027	
40	58.68	7.873	Very high.
45	66.01	9.963	
50	73.35	12.300	A storm or tempest.
60	88.02	17.715	A great storm.
80	117.36	31.490	A hurricane.
100	146.70	49.300	A hurricane that tears up trees, carries buildings before it, etc.

It is seen that before we can proceed to design to resist wind pressure we must decide on the pressure per square foot to be used. Most many-storied buildings are in cities, and municipal building codes specify the pressure to be used. A surprising variation exists in these codes.

Until the adoption of the present code in 1921 the Boston Code read, "Provision for wind bracing shall be made wherever it is necessary." This sentence was followed in the codes of Cambridge, Haverhill, New Orleans and probably other cities. The present code of Boston reads, "All buildings and structures shall be calculated to resist a pressure per square foot on any vertical surface, as follows: For 40 ft. in height, 10 lbs.; portions from 40 to 80 ft. above ground, 15 lbs.; portions more than 80 ft. above ground, 20 lbs." In New York City an assumption of 30 lbs. per sq. ft. is to be made for all buildings in which the height is more than four times the minimum horizontal dimension. In Chicago the requirement is, "All buildings shall be designed to resist a horizontal wind pressure of 20 lbs. per sq. ft. of exposed surface." Philadelphia requires an assumed pressure of 30 lbs. per sq. ft. for buildings erected on open spaces or on wharves. For high buildings erected in built-up districts, not less than 25 lbs. per sq. ft. is to be used at the tenth story, 2½ lbs. less on each succeeding lower story, and 2½ lbs. additional on each succeeding upper story to a maximum of 35 lbs. on the fourteenth story and above. For buildings located where no municipal codes govern, the speaker will quote his own specification: "For buildings not more than 25 ft. to the eave-line the wind pressure shall be assumed at not less than 15 lbs. per sq. ft. and the corresponding normal pressure on the roof. For buildings more than 25 ft. to the eave-line the wind pressure shall be assumed at not less than 20 lbs. per sq. ft. for the side surface above 25 ft. and the corresponding normal pressure on the roof." By normal pressure on the roof is meant that obtained by the Duchemin formula: $\mathrm{Pn} = \mathrm{P}\frac{2 \sin \theta}{1+\sin^2 \theta}$. Fig. 3 gives the pressure on inclined surfaces in accordance with Duchemin's formula.

Let us consider the case of roof trusses resting on walls. By the method of equivalent uniform loading a single stress diagram will answer for any probable combination of dead, wind or snow loads. Every engineer should once and for all verify the correctness of this statement. He can do so by making separate stress diagrams for each condition of loading, both with ends of truss fixed and also with one end on rollers. It is seldom necessary to use rollers for spans under 100 ft. It is possible, though, that wind shear may nearly all come upon one wall due to unequal bending of anchor bolts or to temperature movements.

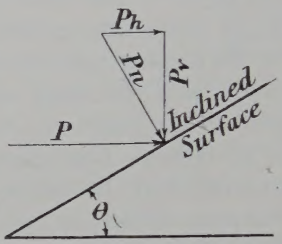


Figure Explanatory of Table.

P = Intensity of pressure on a plane normal to the direction of wind.

P_n = Intensity of normal pressure on a plane inclined at angle θ to the horizontal.

P_v = Vertical component of normal pressure.

P_h = Horizontal component of normal pressure.

$$P_n = P\,\frac{2\sin\theta}{1+\sin^2\theta}$$

$$P_v = P_n\cos\theta = P\,\frac{2\sin\theta\cos\theta}{1+\sin 2\,\theta}$$

$$P_h = P_n\sin\theta = P\,\frac{2\sin^2\theta}{1+\sin^2\theta}$$

Duchemin's Constants for P_n, P_v *and* P_h.

Angle θ.	Slope.	P_n.	P_v.	P_h.	Sec. θ.
4° 46′	1″ to 1′ 0″	0·165 P	0·164 P	0·0131 P	1·003
5° 0′	1$\frac{1}{16}$″ to 1′ 0″	0·173 P	0·172 P	0·015 P	1·004
9° 28′	2″ to 1′ 0″	0·320 P	0·316 P	0·052 P	1·014
10° 0′	2$\frac{1}{8}$″ to 1′ 0″	0·337 P	0·332 P	0·058 P	1·015
14° 2′	3″ to 1′ 0″	0·468 P	0·445 P	0·111 P	1·031
15° 0′	3$\frac{3}{16}$″ to 1′ 0″	0·485 P	0·468 P	0·125 P	1·035
18° 26′	4″ to 1′ 0″	0·575 P	0·546 P	0·182 P	1·054
20° 0′	4$\frac{3}{8}$″ to 1′ 0″	0·612 P	0·575 P	0·209 P	1·064
22° 37′	5″ to 1′ 0″	0·670 P	0·618 P	0·257 P	1·083
25° 0′	5$\frac{5}{8}$″ to 1′ 0″	0·717 P	0·650 P	0·303 P	1·103
26° 34′	6″ to 1′ 0″	0·745 P	0·665 P	0·333 P	1·118
30° 0″	6$\frac{15}{16}$″ to 1′ 0″	0·800 P	0·692 P	0·400 P	1·151
30° 15′	7″ to 1′ 0″	0·803 P	0·694 P	0·404 P	1·158
33° 41′	8″ to 1′ 0″	0·848 P	0·707 P	0·470 P	1·202
35° 0′	8$\frac{3}{8}$″ to 1′ 0″	0·863 P	0·709 P	0·496 P	1·221
40° 0′	10$\frac{1}{16}$″ to 1′ 0″	0·910 P	0·696 P	0·584 P	1·304
45° 0′	12″ to 1′ 0″	0·943 P	0·667 P	0·667 P	1·414
90° 0′		1·000 P	0	1·000 P	∞

FIG. 3. — PRESSURE ON INCLINED SURFACES

Duchemin's Constants for P_n, P_v and P_h

TYPICAL MILL BUILDING BENT

We now come to the common case of a Mill Building Bent with kneebraces (Fig. 4).

The dead and snow loads in the truss of a transverse mill bent are the same as if the truss rested on walls. The columns carry the vertical reactions and the kneebraces are assumed to take no stress. The stresses due to wind loading are statically indeterminate. Owing to the impracticability in ordinary design of calculating stresses in statically indeterminate structures, it is necessary to make certain assumptions and use approximate methods. If the points of contraflexure are located and the horizontal reactions are known a wind stress diagram can be drawn.

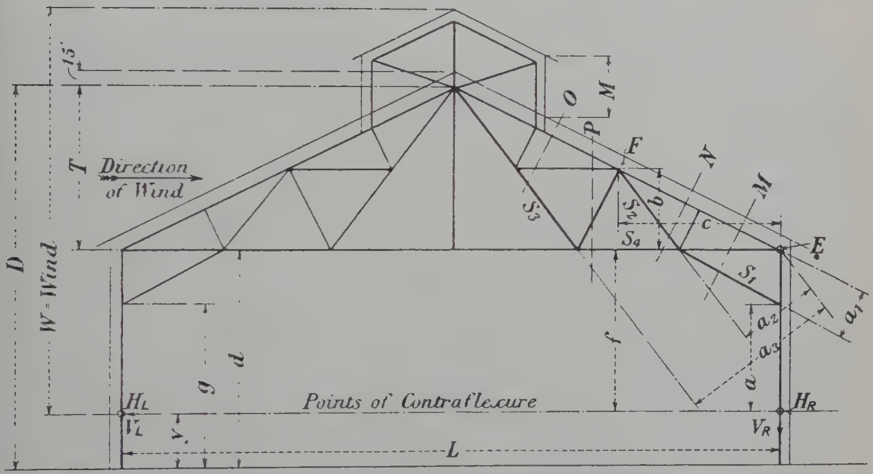


FIG. 4. — TYPICAL MILL BUILDING BENT

For a bent with bottom ends of columns fixed and tops simply supported the points of contraflexure may be determined by the equation $y = \frac{g}{2}\left(\frac{g+2\,d}{2\,g+d}\right)$. For $g = \frac{2}{3}d$, $y = \frac{4}{7}g$; for $g = \frac{3}{4}d$, $y = \frac{11}{20}g$. But the bottoms are never perfectly fixed, which tends to lower the points of contraflexure. At the same time the tops are riveted to the truss which tends to raise the points of contraflexure. As a compromise some engineers assume the point of contraflexure to be midway between the base and kneebrace, or $y = \frac{1}{2} g$. The writer believes that with set anchor bolts, $y = \frac{1}{3} g$ is more nearly correct and uses it in his own work.

Fig. 5 shows the stresses, in a 60-foot bent, for a wind pressure of 20 lbs. per sq. ft. on sides, and 20 lbs., reduced by the Duchemin formula, for roof. The figures in parentheses are due to an assumed uniform

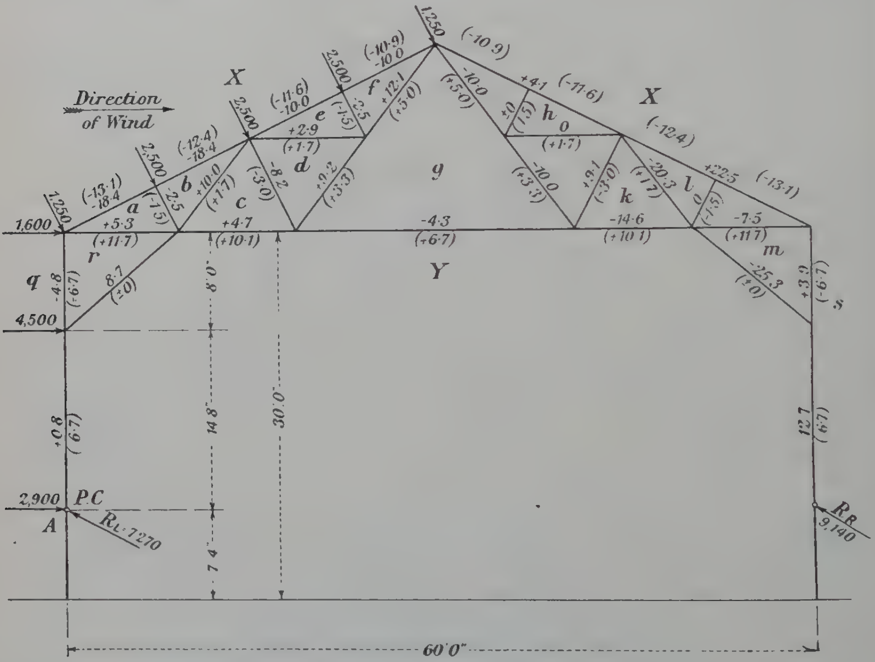


FIG. 5. — WIND AND DEAD LOAD STRESSES FOR 60-FOOT BENT

Slope of roof — 6 inches per foot
Bays — 20 feet
Stresses given in thousands of pounds
— Indicates compression
+ Indicates tension

dead load of 10 lbs. per sq. ft. of roof surface, this being the minimum weight of a roof with sheet metal covering. A wind load of 20 lbs. and dead load of 10 lbs. is about as severe a case as can come upon a mill building. The horizontal reactions are assumed each equal to one-half the horizontal components of the wind loads above the points of contraflexure. It is only the wind loads above these points that produce stresses in the bent; the wind below these points is carried by shear directly to the base of the columns.

Fig. 6 shows a bent of 30 ft. width and 24 ft. height. As may be noticed this is more severe than the 60 ft. bent, even with the wind assumed at 15 lbs. on the sides. Note the reversal of stress on the leeward side. The maximum bending moment in a column according to these

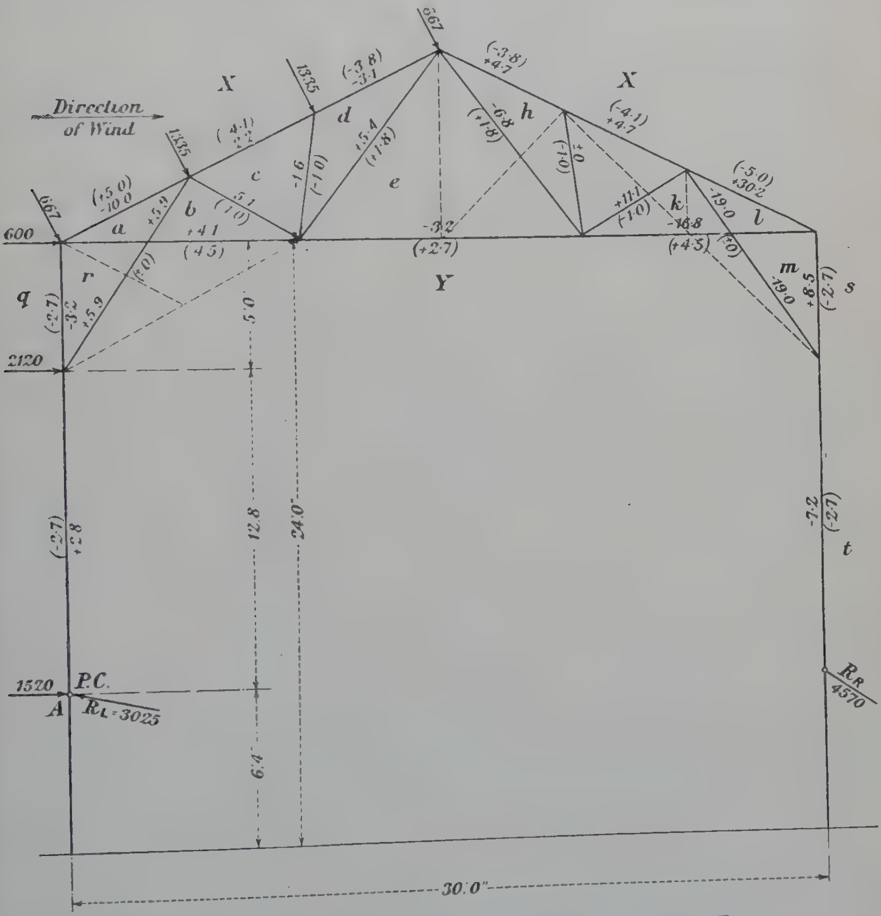


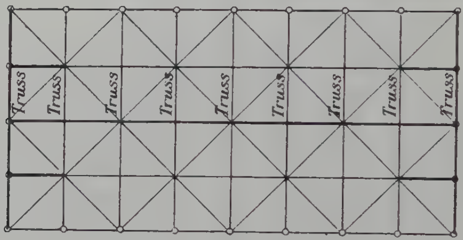
FIG. 6. — WIND AND DEAD LOAD STRESSES FOR 30-FOOT BENT

Slope of roof — 6 inches per foot
Bays — 16 feet
Stresses given in thousands of pounds
— Indicates compression
+ Indicates tension

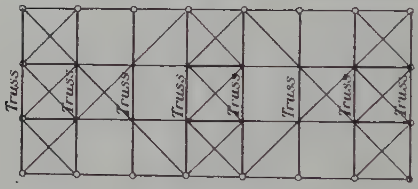
assumptions is at the foot of the kneebrace on the leeward side. More buildings are weak here than in any other place.

In buildings with traveling cranes, kneebraces can seldom be used. With the gusset plate connecting truss to column made as large as the crane clearance will allow, it is often necessary to obtain lateral stability elsewhere than wholly from the transverse strength of the bents. An effective way, when it can be done, is to transfer all transverse thrust to the ends of the building by means of bracing in the plane of the bottom chords and thence by diagonals to the ground. Fig. 7 shows different ways of doing this.

BRACING-PLANE OF BOTTOM CHORDS



BRACING-PLANE OF BOTTOM CHORDS.



"TABLE" BRACING-PLANE OF BOTTOM CHORDS

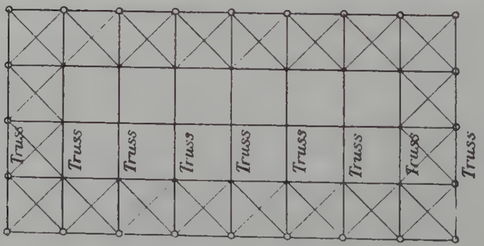


FIG. 7. — BRACING — PLANE OF BOTTOM CHORDS

For a span of 80 ft. and an even number of bays the upper figure is ideal. A span of 60 ft. and an odd number of panels is more difficult to handle. The middle figure is the consensus of the opinion of six structural engineers all well versed in designing buildings. The lower figure is the "table" bracing system, a favorite method of the late P. L. Wolfel.

Bracing systems in the plane of the bottom chords have their limitations. It is advisable to have the transverse bents strengthened for rigidity in addition to a system of bracing in the plane of the bottom chord. Stiffness and rigidity are necessary as well as strength to resist wind force. Provision should be made against vibration due to traveling cranes or other moving loads.

Certain points applicable to wind bracing of all buildings follow:

(1) Angles or other stiff bracing should be used in the plane of the bottom chord; rods are usually used in the plane of the top chord.

(2) It should be remembered that wind blows on the ends of buildings as well as on the sides.

(3) Trusses resting on walls should be firmly anchored to resist any uplift from suction or pressure on the under side of the covering.

(4) The anchorage, including anchor bolts and their connections, should be designed to resist the bending moments of the base of columns.

The detailer plays a far more important part in making a design efficient than that for which he is usually credited. Especially is this true of the provision he makes for the connections of kneebraces, and bending moments of columns. An excellent design may be weakened by poor details.

Wind Bracing for Modern High Buildings

We next come to the modern high buildings of skeleton construction and light curtain walls, with its problem of determining wind stresses and making proper provision to resist them. The wind stresses assumed and the stresses resulting therefrom often play an important part in the design of a building especially when the width is small compared with its height.

It is evident that for horizontal wind pressure a transverse bent of the structure is a cantilever beam or truss, with its fixed end at the foundation. (The walls, partitions and fireproofing add stiffness to the building and resist a portion of the wind pressure, but for present purposes this is not considered.) The architects of early high office buildings developed four types of wind bracing (Fig. 8): vertical sway rod type; lattice girder type; portal type; and kneebrace type. Each type served its purpose, but because of their limitations they have all largely given way to the gusset plate type, — a system of wind bracing without diagonals.

Different methods of calculating wind stresses in high office buildings have been developed in late years. A broad classification may be (1) exact methods and (2) approximate methods. None of the exact methods are followed in practice, at least not to the knowledge of the writer. With all the exact methods the stresses are statically indeterminate and are not workable in everyday use. The one nearest to being so is the slope-deflection method of Wilson and Maney.

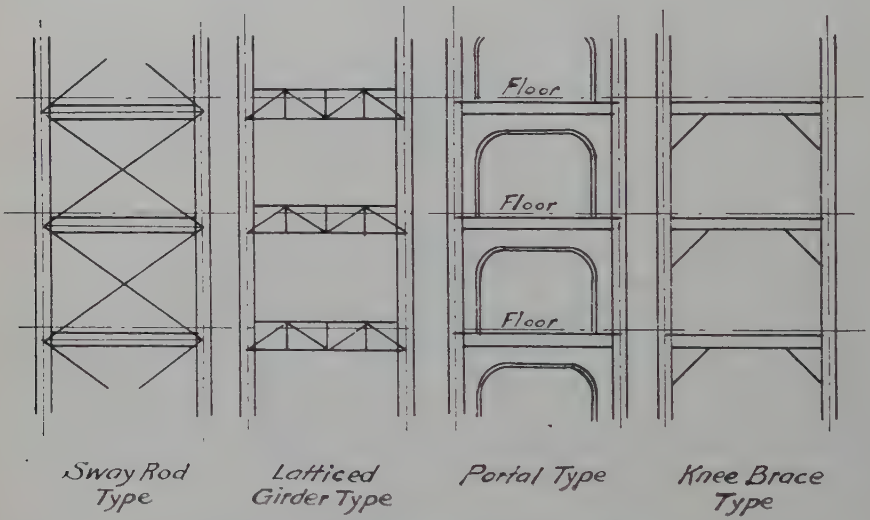


FIG. 8. — EARLY TYPES OF WIND BRACING

These have been superseded to a large extent by the "gusset plate" type

For practical purposes resort must be had to approximate methods. Three of these methods will be given, considering a single bent. It will be assumed that all columns in any given story have the same sectional area and the same moment of inertia, and that the joints are perfectly rigid. The adjacent joints occupying the corners of a rectangle will after distortion occupy the corners of an oblique parallelogram. It is assumed that the point of contraflexure of each column is at midheight of each story. Let it be stated here that in none of the methods proposed are the assumptions strictly consistent.

Method I may be called the Cantilever method (Fig. 9). A section or bent of the building is considered as being similar to a beam loaded

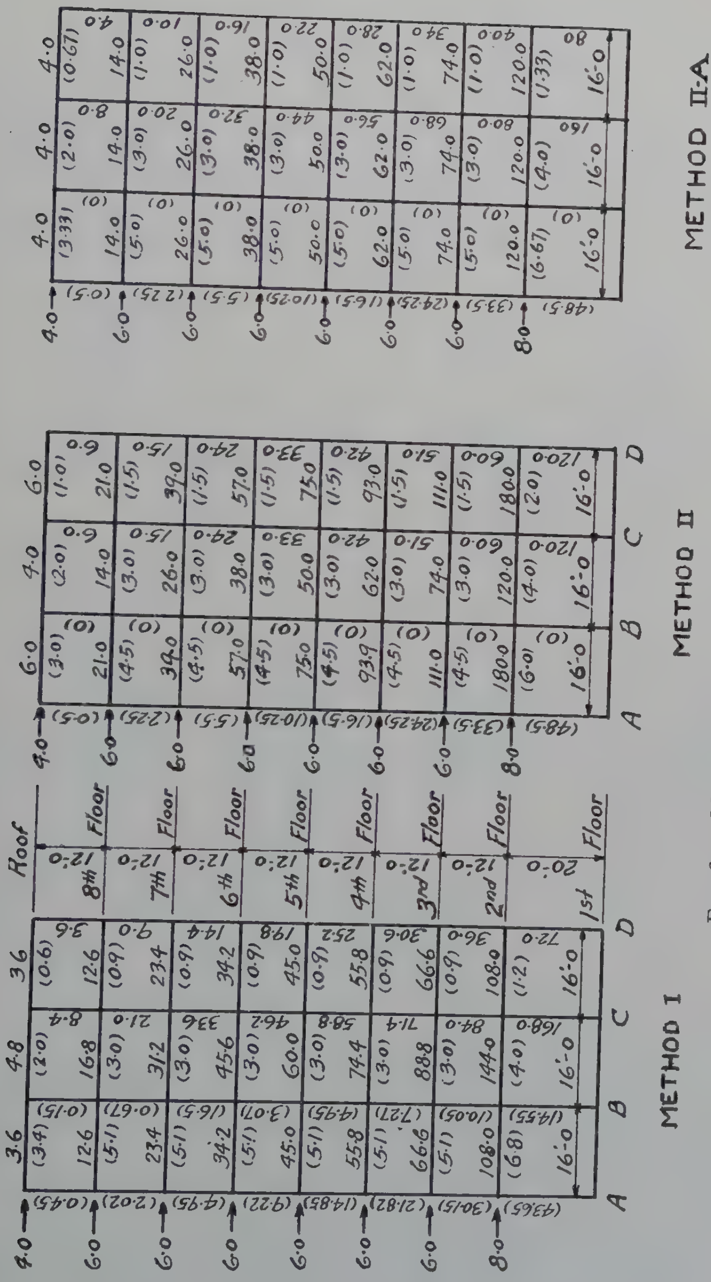
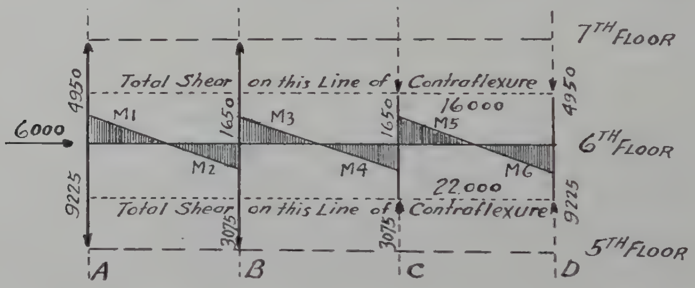


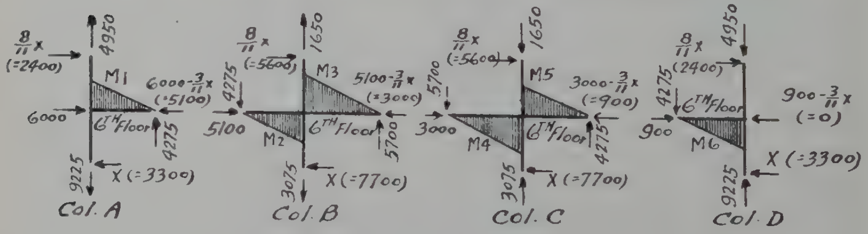
FIG. 9. — METHODS OF CALCULATING WIND STRESSES

I — Cantilever method; II — Method of equal shears; II-A — Portal method as a cantilever. If the beam had a solid web it would be possible to find the internal stresses at any point.

If, however, rectangles be cut out of the beam between the loads there will then be a different condition of stress. What was the horizontal shear of the beam will now be a shear at the point of the contraflexure of the floor girders, causing bending, and as in the beam, the nearer the vertical axis the greater the shear. The vertical shear in the beam would be taken up by the columns as a shear at the points of contraflexure, and the amount of this shear taken by each column would, as in the beam, increase toward the vertical axis. The direct stresses of tension or compression in the beam would act on the column as a direct load of either tension or compression, and as in the beam would decrease toward the vertical axis.



COLUMN SHEARS AND GIRDER MOMENTS AT SIXTH FLOOR



SIXTH FLOOR JOINTS OF BUILDING FRAME
METHOD I

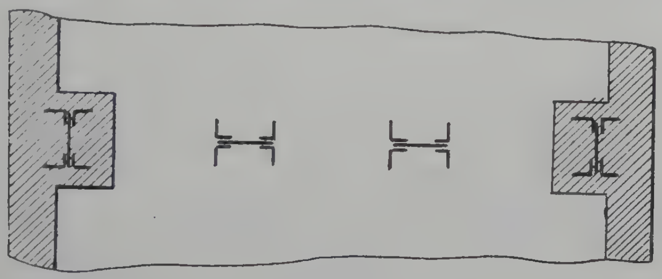
FIG. 10. — COLUMN SHEARS AND GIRDER MOMENTS — METHOD I

Each intersection of column with floor girders would be held in equilibrium by forces acting at the points of contraflexure; and to find all the forces acting around a joint at any floor the bending moments of the building at the points of contraflexure of the columns above and below the floor in question are found.

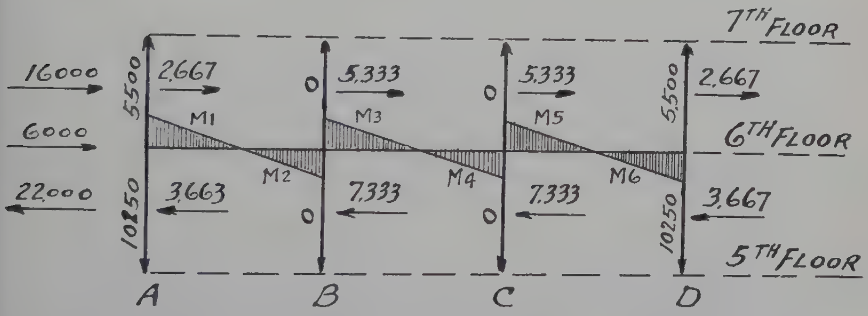
Method II (Fig. 9) may be called the method of Equal Shears. It is assumed that the horizontal shear on any plane is equally distributed among the columns cut by that plane. By this method the bending moments in columns are equal. The exterior and interior columns may be so turned as not to be able to resist equal bending moments.

The column shears and the girder moments of Method I are shown in Fig. 10.

Method II–A (also shown in Fig. 9) is a special case of Method II and may be called the Portal method. The total horizontal shear along any plane is divided by the number of aisles instead of by the number of columns as in Method II. An outer column thus takes but one-half the shear of an interior column. For equal spacing the direct or vertical axial stress due to the overturning moment of the wind is all taken by the outside columns as in Method II. It is noted that the bending moment in an outer column is one-half that in an interior column; that the point of contraflexure of each girder is at its midlength; and that the bending moments due to wind are alike for all girders on the same floor



CROSS SECTION OF COLUMNS IN TRANSVERSE BENT

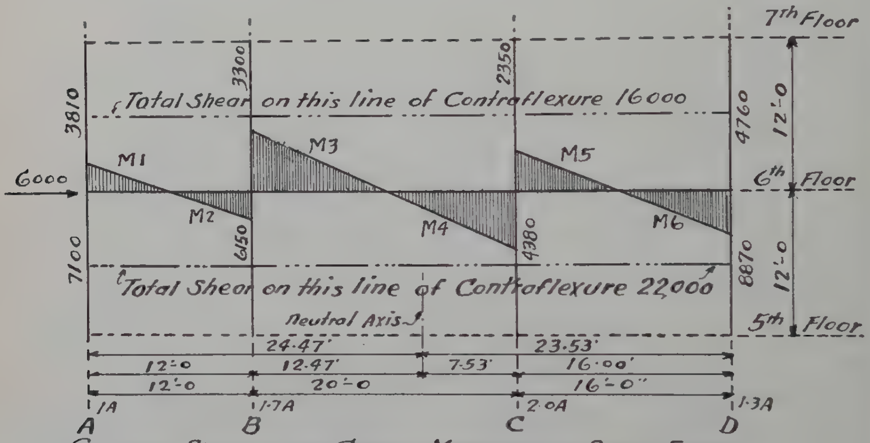


COLUMN SHEARS AND GIRDER MOMENTS AT SIXTH FLOOR
METHOD II A

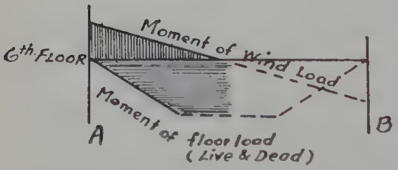
FIG. 11. — COLUMN SHEARS AND GIRDER MOMENTS — METHOD II–A

of any transverse bent. This is an ideal condition for the detailer and the shop. The designer finds this method very simple and his work easily checked. The bending moment in a girder is the sum of the moments in the outer columns above and below that girder.

Fig. 11 shows the column shears and girder moments at the sixth floor according to Method II–A, also a cross section of columns in a transverse bent where this method is especially applicable.



COLUMN SHEARS AND GIRDER MOMENTS AT SIXTH FLOOR
UNEQUAL BAYS, VARYING SECTIONAL AREA



GRAPHICAL COMBINATION OF MOMENTS FROM VERTICAL LOADS AND WIND LOADS IN FLOOR GIRDER

FIG. 12. — UNEQUAL SPACING OF COLUMNS AND VARYING MOMENTS OF INERTIA

Attention is called to unequal spacing of columns and varying moments of inertia. An illustration is given in Fig. 12. To bring all direct stresses to the outside columns, the wind loading taken by each bay should be assumed proportional to its span. Horizontal shears in columns are proportional to their respective moments of inertia. At the bottom of this sketch will be noted a diagram showing how the moments from vertical loads and wind loads are usually combined.

Choice of Methods

Each of the foregoing approximate methods has its advocates. The writer followed Method I in designing the twenty-story Finance Building in Philadelphia, and Method II–A for the eighteen-story Hurt Building in Atlanta. He believed that Method I is more in accordance with the true distribution of stresses, but because II–A simplifies and duplicates connections, he is inclined to use it.

It is not always practicable, or even advisable, to hold to one system of bracing throughout. A considerable portion of wind pressure may sometimes be carried to the ends and sides of a building and thence transferred to the foundations by means not permissible in interior bays.

A building like the Fuller, the Singer, the Woolworth, the Metropolitan Tower, in New York City, is each a problem by itself. For unusual structures like these, careful study must be made and special methods often devised.

Working Stresses

In conclusion, a word about working stresses should be included. Because wind loads are intermittent and seldom reach their maximum, greater stresses are permitted than for roof, floor and dead loads. The New York and Chicago codes allow for the combined stresses due to dead, wind and other live loads an increase of 50 per cent in the working stress over that allowed for dead and other live loads alone, provided the section thus obtained is not less than that obtained if wind force be neglected. This is equivalent to neglecting wind stress when it does not exceed 50 per cent of the stress due to combined dead and other loads. The Boston code allows an increase of but twenty per cent, which in the opinion of the writer is needlessly severe.

DESIGN OF DETAILS IN TIMBER STRUCTURES

By Ray H. Lindgren,* Member, Boston Society of Civil Engineers

(Presented before the Designers Section, Boston Society of Civil Engineers, on April 14, 1926)

Although timber has been superseded to a considerable extent by steel and concrete, especially in permanent work, it is still one of the most widely used structural materials, and while its use for such purposes dates back into the dim and distant past, there is today a surprising lack among engineers of standards of timber construction, particularly regarding details.

The old-time bridge engineer was a master of the art of timber framing, and while much of his work was the result of experience and judgment rather than any definite analysis of stresses, his structures stood the test of time. The results of his experience are today represented by the standard designs for bridges and trestles now filed away among the archives of the various railway engineers' offices.

The old-time ship carpenter, too, was a master craftsman, but there are today relatively few surviving members of this ancient craft, as was only too well demonstrated during the late war.

Outside of these two groups of men there is at present very little knowledge of the art of timber framing, and in many cases a lack of appreciation of the necessity for such knowledge. In many engineering schools the instruction in timber design consists of a few problems in computing the sizes of beams and in solving the stresses in a roof truss with the resultant computation of the sizes of its members. Usually little if any attention is given to the method of framing, with the result that the feeling among many young engineers is that the carpenter is fully capable of designing the joints if given the sizes of the members.

The method of finding the stresses in a structure is of course the same whatever the material, and the details in a timber structure should be analyzed for strength as much as though the structure were of steel or concrete. As a matter of fact, the design of the joint often determines the size of member required. The details made by the so-called

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* Designing Engineer, The Crandall Engineering Company, 102 Border Street, East Boston, Mass.

"practical man," who has only a hazy idea of the theory of structural mechanics, are often highly impractical and sometimes absolutely dangerous due to his lack of conception of the problem involved. The designer should give as much, or more, attention to the details of timber construction as he would to steel or concrete, and he has greater opportunity for originality, since there are comparatively few standards to which he can refer.

The determination of the allowable unit working stresses to be used in the design of timber structures is difficult, since it is possible to secure a different answer from almost every source consulted. Of course, in cases which are governed by the building laws no latitude is allowed the engineer. In other cases he must consider whether the work is of a permanent or temporary nature, the grade of lumber used, the possibilities of deterioration during the life of the structure, the degree of exactness with which he knows the stresses involved, and the possibilities of heavier loads than those figured. In other words, the engineer must determine the factor of safety desired and apply it to the ultimate strength, remembering that timber is the product of organic growth and liable to various defects due to its nature. The working stresses given by the American Railway Engineering Association Manual of 1921 are perhaps as widely known as any and give conservative results.

In this connection the allowable working stresses in compression perpendicular to the grain are worthy of further consideration. The A. R. E. A. Manual gives these working stresses on inside work as Douglas Fir, No. 1 structural, 350 lbs.; southern yellow pine (dense), 350 lbs.; and oak, 500 lbs. It would be only in the finest selected pieces of live fir timber and very ordinary pieces of yellow pine that any such comparison could be made. On the lumber commonly used in this section of the country the following figures would be more suitable, based on the value of oak at 500 lbs.: yellow pine (dense), 350 to 400 lbs.; Douglas fir, 300 lbs. And in wet locations the variation would be more marked, as the bearing value of the fir decreases much more rapidly than that of the pine. It may also be noted that these allowable pressures in compression perpendicular to the grain are the ones most frequently exceeded in ordinary practice. Every bolt so placed as to develop its full tensional value, and carrying the standard size square plate washer, is exceeding the allowable value of side grain compression, to say nothing of the results when only the ordinary round punched washers are used.

The old question of allowable pressures on inclined surfaces next arises. There are two formulæ in general use, — Jacoby [*] and Howe.[†] The Jacoby formula is based on the resolution of the inclined force into its two components, — perpendicular to and parallel with the grain. The Howe formula is based on a constant indentation. Fig. 1 shows these two formulæ.

It is evident that Howe is much the more conservative of the two, as his curve is below that of Jacoby throughout the whole range of 90°.

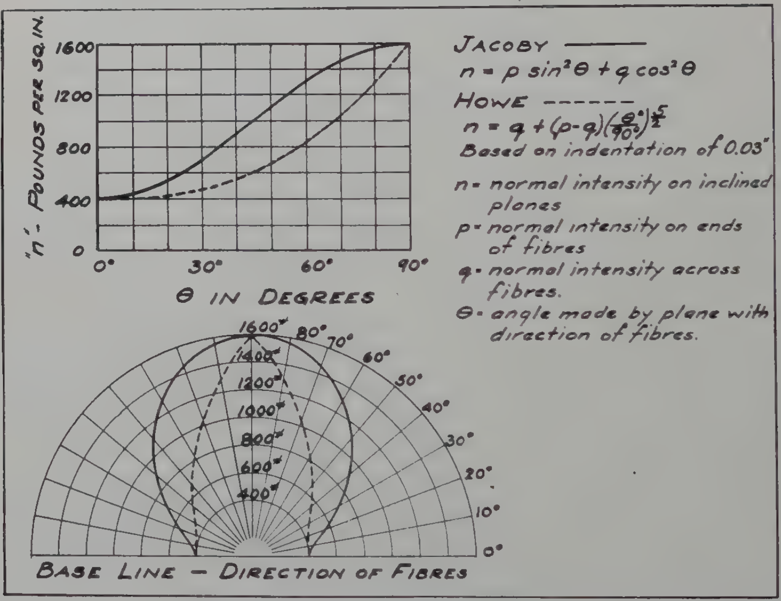

FIG. 1. — JACOBY AND HOWE FORMULÆ

The real fallacy of the Howe formula becomes apparent when it is realized that indentations produced by stresses well inside the elastic limit for pressures perpendicular to the grain could only be produced by stresses approaching the ultimate for pressures parallel with the grain. The idea of basing the allowable stress on the indentation is an admirable one, but it would seem necessary, before we do that, to decide on the allowable indentation at the two extremes of direction, and also on the variation we are to allow through the intermediate angles. This offers a very fertile field for further experimentation.

The Jacoby formula is in very common use for the coniferous woods, and is the one used by the writer. It is given by Professor

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[*] H. S. Jacoby, "Structural Details."

[†] M. A. Howe, Engineering News, Vol. 68, Nos. 5 and 10.

Swain in his "Fundamental Properties of Materials," with the comment that "if the reader thinks that the surface could not safely sustain simultaneously the two forces (components) then he may adopt some empirical rule more satisfying to his instinct."

The lower set of curves shows a very convenient form for ready use of these formulæ. If the curve be drawn on tracing cloth or paper, and the base line placed parallel to the fibres of the member with the origin on the cut, the line of the cut prolonged will give the allowable stress on that surface directly. These curves may be made up with any combination of allowable stresses desired.

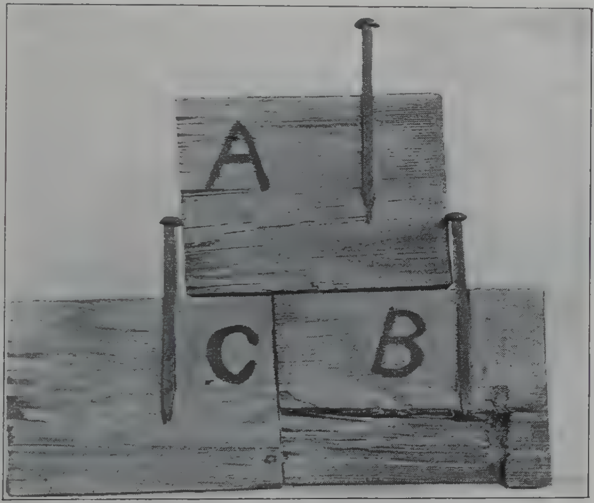

FIG. 2. — HOLDING POWER OF SHIP SPIKES

A — Bored with $\frac{1}{2}$ in. bit — 4 in. deep
B — Bored with $\frac{7}{16}$ in. bit — 4 in. deep
C — Bored with $\frac{1}{2}$ in. bit — 1 in. deep
All 8 in. × $\frac{1}{2}$ in. ship spikes driven 5 in. into wood

In steel fabrication all bolting and riveting is usually done with holes punched 1⁄16″ large. In timber work this is not the case. All bolting and spiking should be done with holes bored either the same size as the bolt or with a drift, depending on conditions. If there is any necessity for the bolt to develop any resistance to withdrawal it is necessary for the hole to be bored slightly smaller than the nominal size of the bolt, or, as we say, with a "drift." Fig. 2 shows pictures taken of tests performed by The Crandall Engineering Company to investigate the different values of the holding power of ship spikes according to the way in which the holes were bored. "A" shows a spike driven in a hole of the same diameter as the nominal size of the spike. A very few of the fibres are bent down except at the point. In "B" a similar spike was driven in a hole 1/16″ undersize. All the fibres in contact with the spike are bent down but are not broken off. In "C" a similar spike was driven in a hole bored neat size for one inch in depth. All the fibres are bent down sharply and the majority of them broken off. It seems needless to state that the results of the tests ranked the holding power in the following order: "B," "A," and "C." This also shows that there is another reason for boring for spikes besides the practical one of avoiding splitting the timber.

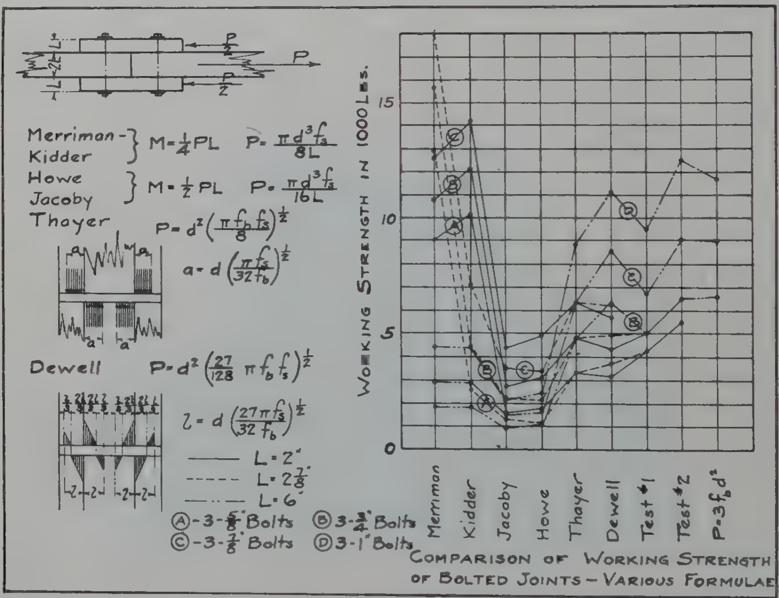

FIG. 3. — COMPARISON OF WORKING STRENGTH OF BOLTED JOINTS

Nowhere is there a greater variation of formulæ than in the value of the lateral resistance of bolts in bolted timber joints. Fig. 3 shows the various ones enumerated by Dewell in his book on "Timber Framing." These have been rearranged to give directly the value P of the bolt in double shear. P and L are as shown in the diagram.

f_b = allowable stress in bearing on the wood.
f_s = allowable extreme fibre stress in bending of the bolt.
d = diameter of the bolt.

The first two formulæ are derived by considering the bolt as a restrained beam with the bending moment as indicated. Up to certain thicknesses of timber no bending is considered, the value being dependent wholly on shear and bearing, which is, of course, independent of the formulæ given.

Thayer's formula* is based on the assumption that the bearing is uniformly distributed over a distance a such that with the bearing at the allowable fibre stress, the resulting bending moment will develop the allowable fibre stress of the bolt in bending.

Dewell's formula is based on a theory similar to that of Thayer except that he assumes a triangular distribution as shown.

The curves at the right are an attempt to show graphically the comparison between the working strength of nine different joints as figured by different formulæ. The values of the different joints are plotted on the vertical line assigned to each formula and the various values for the same joint are connected to form a curve. Tests No. 1 and No. 2 are from reports given by Dewell for actual tests made by him in 1915. The first is one-half the load producing a slippage of 1⁄16″, and the second is that producing a slippage of 0.025″. The last formula is one adopted by the Crandall Engineering Company as a close approximation for ordinary conditions. It is based on a bearing over a length 1½ times the diameter of the bolt.

The formulæ of either Thayer or Dewell will give satisfactory results, but Dewell must be used with considerable caution, for a special variation in the formula must be used, when l is less than L, while in Thayer a is very seldom less than L. For ordinary conditions of work the formula $P=3f_bd^2$ is simple to use and gives satisfactory results.

One of the most common details occurring in timber construction is the splice. There are many and varied kinds possible, but Fig. 4 shows the principal ones. "A," "B," and "C" are typical compression splices. "A" is the ordinary "halved" joint, good in pure compression but of small value under flexure. "B is the so-called "scarf" joint, not as good as "A" for straight compression, but better under flexure, in which case it is usually made somewhat longer. "C" is the ordinary "butt" joint, the best of all for straight compression. In all of these the bolting simply serves to hold the two sections together and in line with one another and cannot be figured in any way.

"D," "E," and "F" are typical tension splices. "D" is the "bolted and keyed" joint which appears to the writer to be the most satisfactory tension joint which can be made up without unduly expensive labor costs. "E" is the "steel tabled fish plate" splice, a very effective type, but requiring exceedingly careful workmanship by good carpenters and careful inspection to insure good work. "F" is the ordinary "bolted fish plate" splice, very easy to make up, but when used with large loads requiring an excessive amount of bolting. The three splices shown have all been designed to the full strength of a 10″×12″ yellow pine timber having a net section of 9″×10″ or 90 sq. in., which at 1,600 lbs. per sq. in. gives a total value in tension of 144,000 lbs.

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* H. R. Thayer, "Structural Design."

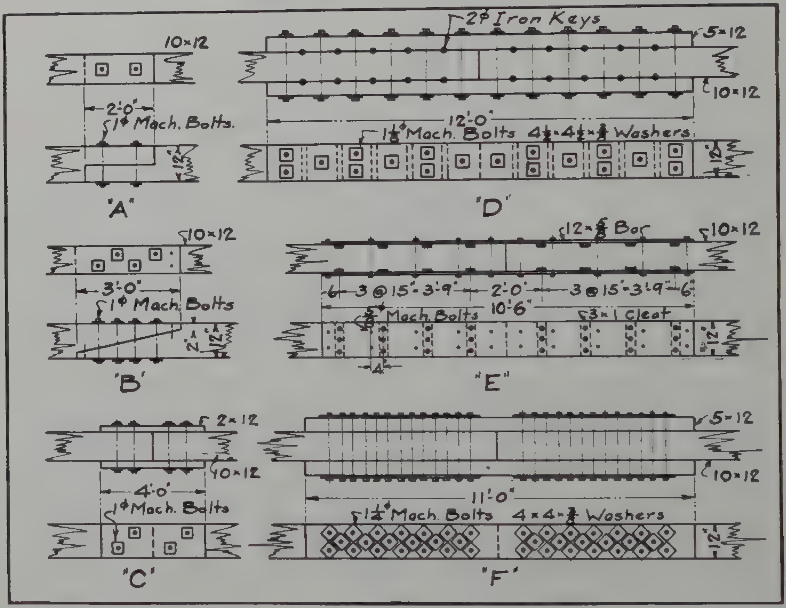

FIG. 4. — PRINCIPAL TYPES OF SPLICES

"D" requires splice pads half the size of the main member or 5″×12″. Using 2 inch iron keys and taking the allowable average unit pressure on the projected area of the pin at $p' = \frac{2}{3}p + \frac{1}{3}q$ or 1,200 lbs. per sq. in., the value of each key is 1,200×12×1 = 14,400 lbs., or a total of 10 keys is required each side of the butt. The spacing of the keys must be sufficient to allow for ample strength in shear parallel to the grain, and sufficient bolts must be provided to carry in tension one-half the total load on the splice, or 72,000 lbs. Using 1⅛ in. circular bolts, 7 bolts are required. At 400 lbs. per sq. in., 27.75 sq. in. will be required in bearing for the washers. The use of 4½″×4½″ having an area of 24.25 sq. in., or a stress of $\frac{11{,}104}{24.25} = 458$ lbs. per sq. in., will be satisfactory. In making up this joint the splice plates are first clamped on, the holes for the bolts bored, and the bolts then set up tightly. Next, the holes for the keys are bored, using 1/16″ drift, taking care to get the auger centered on the seam and that the hole does not run off to one side. Then the pins are heavily coated with tar or some of the asphaltic preparations and driven home. Care must be taken, if unseasoned lumber is used, that the bolts are occasionally tightened until all shrinkage has taken place. This is the only disadvantage I know of in this joint, which we are now using wherever local conditions do not prevent.

In "E" using cleats 1″ thick, each has a value of $1{,}600 \times 12 \times 1 = 17{,}200$ lbs., and 8 cleats are required on each side of the butt. Using three 7/8″ rivets the net width of the plate is 9″, and at 16,000 lbs. per sq. in. a minimum thickness of ½″ is necessary, but 5/8″ will be used. The spacing of the cleats must be sufficient to allow for ample strength in shear parallel to the grain. Bolts must be provided, too, for the couple acting on the splice plate at each cleat, $\frac{17{,}200 \times \frac{13}{16}}{4} = 3{,}900$ lbs., and two 5/8″ bolts will be used. The principal disadvantage of this splice is the careful workmanship necessary to secure an even bearing on all of the cleats without which the value of the splice may be greatly impaired.

In "F" we require splice plates of half the size of the main members or 5″×12″. Using 1¼″ bolts and the formula $p = 3f_b d^2$ previously mentioned, the value of each bolt is 7,400 lbs., and 20 bolts are required on each side of the splice. The bolts must be spaced to maintain the net section and also not to exceed the allowable stress in shear parallel to the grain. This joint very strongly resembles a piece of armor plate and is not recommended for use in splices as highly stressed as this one. The bolts should be given 1/16″ drift to insure good bearing throughout. Steel plates are sometimes used on the sides instead of wood, but this is a serious mistake as it is an utter impossibility to bore holes through from one plate to the other and hit the hole on the far plate perfectly each time, and unless this is done the value of the bolt in bearing on the timber is lessened to a great extent.

Figs. 5 and 6 show the joint details of the transverse frame of a floating dock. No fastenings are shown, as the joints are all in compression and the fastenings are not counted on to carry any of the loads but serve mainly to bind the members together. The allowable stresses on the inclined planes are figured according to the Jacoby formula as

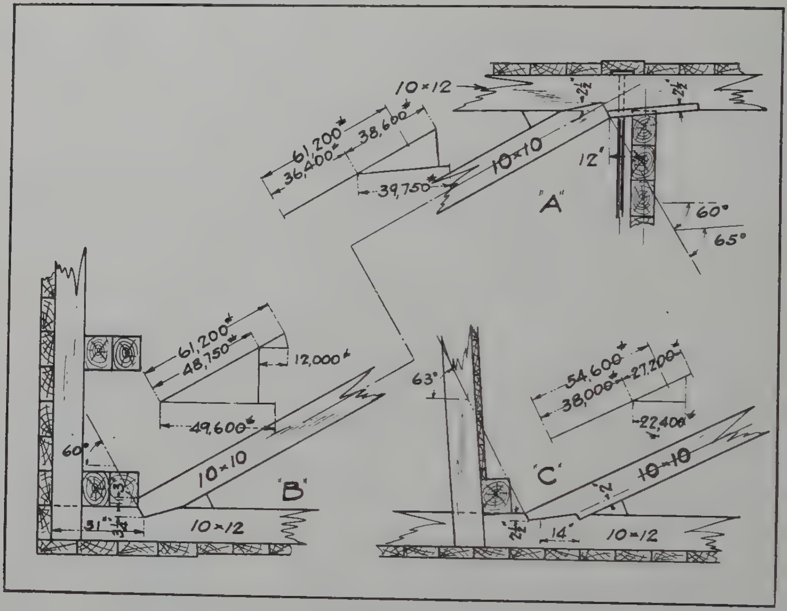

FIG. 5

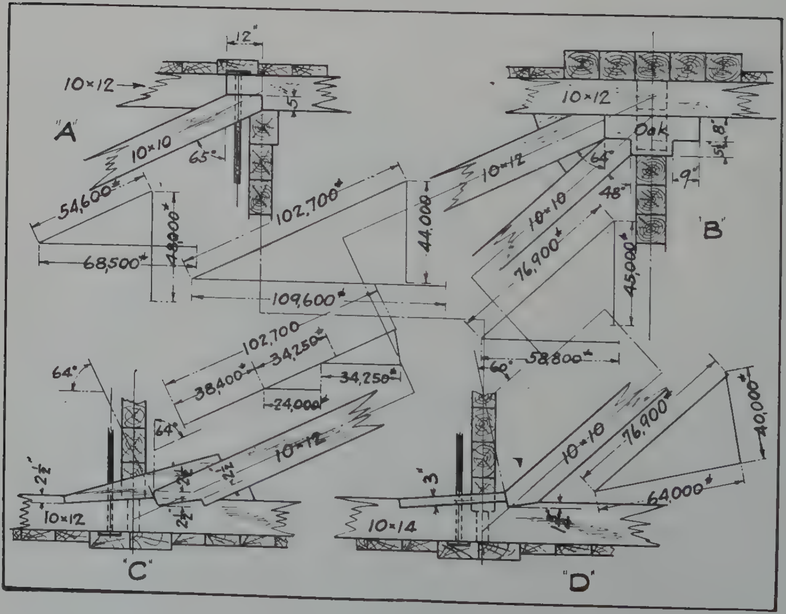

FIG. 6

JOINT DETAILS OF TRANSVERSE FRAME OF FLOATING DOCK shown on Fig. 1. The primary requirements of such joints are that the framing be made as simple as possible, involving the minimum number of cuts; that wherever possible the cuts shall be so arranged that one member may be dropped directly into place in the other, and the two members "saw cut" into bearing. In other words, the carpentry work must be simplified as far as is consistent with good workmanship and the stresses held strictly within the allowable limits.

In joint "B" we have an axial stress of 61,200 lbs. in the diagonal. Making the end cut 3¼″ deep vertically we have an actual area on the inclined surface of $3\frac{3}{4} \times 10 = 37.5$ sq. in. This cut is practically normal to the inclined member and makes an angle of 60° with the bottom chord. From the curve shown on Fig. 1 we see that the minimum allowable stress is that at 60° or 1,300 lbs. per sq. in., which gives $37.5 \times 1{,}300 = 48{,}750$ lbs. acting axially along the diagonal. This force may also be limited by the value in shear parallel to the grain at this point. There are 31″ available to the end of the chord, so we have $31 \times 10 \times 160 = 49{,}600$ lbs. allowable, which is seen to be more than the horizontal component of the above force. The 3″ bearing on the side of the longitudinal member is limited by the allowable side grain pressure, or $3 \times 10 \times 400 = 12{,}000$ lbs. This force is, of course, horizontal, and projecting upward normal to the remaining face of the cut we find that this forms a perfect closure of the force diagram. This last force is so obviously within the allowable that it is needless to figure it. These forces all combine to act axially along the member so there is no appreciable eccentricity. The longitudinal member must have sufficient fastening to carry the 12,000 lbs. horizontal force into the bottom chord.

Joint "A" at the other end of the same member carries, of course, the same load of 61,200 lbs. That part of the end which rests in the cut in the top chord has an area of $2.8 \times 10 = 28$ sq. in. at an allowable stress of 1,300 lbs. per sq. in. on the 60° plane, making the total allowable stress on this portion 36,400 lbs. acting as shown. That portion resting against the scab has the same area, but a slightly larger allowable stress, 1,380 lbs. per sq. in., due to the greater angle which the cut makes with the plane of the scab, giving a total allowable stress on this portion of 38,600 lbs. This latter force may be limited by the allowable pressure on the end of the scab against the top chord which is seen to be more than the component acting in that direction. It is evident that this joint is good for considerably more than the actual load to be carried. This joint is subject to a slight eccentricity, due to the fact that the end pressure is not axial, but this is more than countered by the frictional resistance of the end against slipping aside from the value of the fastenings. This joint is very simple in construction and may be readily "saw cut" to a perfect bearing, a very desirable feature.

In joint "C" we have a "double nib" joint. This is not as desirable as the "single nib" because of the difficulty of fitting to secure an even bearing on each nib and the impossibility of "saw cutting" both to a bearing. The calculations are as indicated, the value of the second nib being limited by the value of the horizontal component in shear parallel to the grain. This joint has practically no eccentricity.

On Fig. 6 joint "A" shows the other end of this member. This joint is rather out of the ordinary in that we have not only framed in the diagonal but also formed a butt joint in the top chord. As shown, the allowable horizontal and vertical stresses of this joint are more than ample. In addition, there are side plates over this joint which are not shown here, forming a bolted and keyed joint for the top chord similar to that shown on Fig. 4.

In joint "C" we have a double nib joint used in combination with a scab. This has the disadvantages of the double nib joint, but the use of the scab gives the value of three nibs without the disadvantages of attempting to fit the three all in one member. This is figured as shown on the diagram, the value of the second nib being limited by its value in shear parallel to the grain as shown. The allowable stress against the scab is 2.5 × 10 × 1,370 (p at 64°) or 34,250 lbs. acting horizontally. The closing line is, of course, perpendicular to the remaining side of the joint, which makes a practically perfect closure. The stress on this surface shows by inspection to be well within the allowable. The scab in this joint is carried up over the lower end of the inclined member to secure better fastening, but the inspector must be sure that this piece is cut with the grain parallel to the bottom of the scab itself, or the value of this piece will be limited by the longitudinal shear along the grain. This joint has a slight eccentricity, but the effect is negligible. This joint illustrates one point mentioned earlier, — that the size of the diagonal had to be increased from 10″ × 10″ to 10″ × 12″ to provide sufficient joint strength.

Joint "D" is a single nib joint used in combination with a scab. In this case, on account of the steep inclination, the cut could not be made normal to the diagonal, so it was made to go in with one saw cut through scab. In this case the allowable stresses on each of the two surfaces are plotted normal to their planes and give an approximate closure. The value of that portion of the cut which comes on the scab might be limited by the value of the scab at the other end, but it is evident that such is not the case here. The forces on this joint are approximately axial, so there is very little eccentricity.

Joint "B" shows the upper end of the members in joints "C" and "D." The key piece is made of oak to take advantage of the larger value of this wood in side grain bearing. As will be seen, the lower member is slightly deficient in the vertical component, so wedges have been added between the members to make up for this deficiency.

In all these joints the depths of the cut which it is possible to make into the bottom chord have been strictly limited, since this is a tension member, and for this reason more elaborate joints have been used than would at the first glance seem necessary. The splice in the bottom chord is very similar to "D" on Fig. 4. There is an eccentricity on the bottom chord due to the fact that the points of application of the horizontal components do not coincide with the center of the net section, but this is taken care of partly by offsetting the vertical tension rods and partly by the depth of the joint.

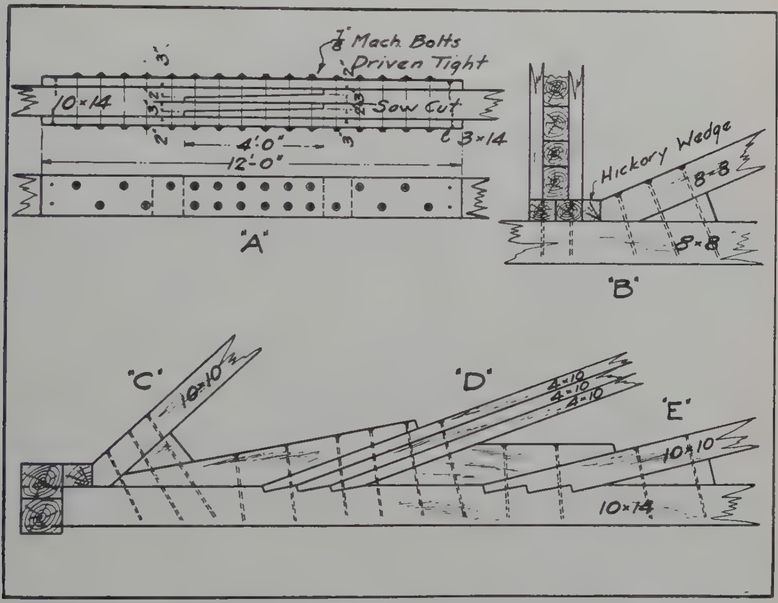

Fig. 7. — Examples of Poorly Designed Joints

In Fig. 7 an attempt has been made to show a few undesirable features which the writer has encountered. Joint "A" is a very ingenious if not very desirable tension joint. Allowing liberal values this joint has an allowable value of 77,175 lbs. as against the 144,000 lbs. used in figuring the tension splices on Fig. 4. In addition to this the labor involved in this splice is much more than on any of those shown, and the probability of securing good bearing on the center bolts throughout their length is rather dubious.

Joint "B" shows a very common failing in joint design, especially by the untrained man. This designer is not satisfied to allow his diagonal to come down and fit against the longitudinal member, but uses a hickory wedge to insure an absolutely tight fit, and then places his whole reliance in five drift bolts, only two of which are beyond the wedge, to take the horizontal component.

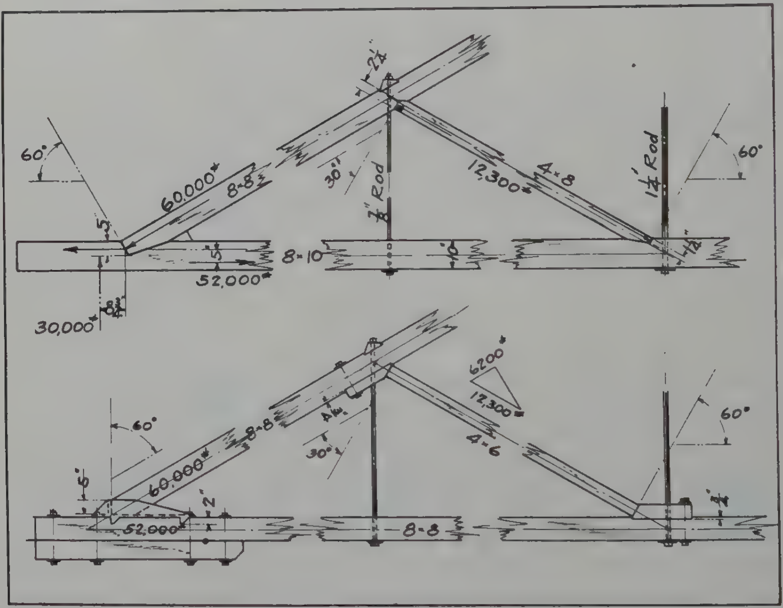

Fig. 8. — Typical Roof Truss Joints

In joint "C" we have the questionable side grain bearing again in evidence. If for no other reason than deflection, it is advisable to avoid such details, since the indentation is so much greater on this plane than the one at right angles to the fibres that the deflection is materially increased by such joints. "D" and "E" show three nib joints, a detail which should never be used if possible to avoid it, as the labor of fitting is greatly increased and the value of the bearing on the inner nibs is very questionable. Of course, in the case of the laminated member this is practically eliminated, but in any case the value of the inner nibs is limited by the shear parallel to the grain, which feature must be always kept in mind.

In Fig. 8 are shown a few typical roof truss joints framed in two different ways, first by direct cuts and second by the use of scabs and cast-iron shoe. It may be noted that the first method necessitates a larger size for two of the members as well as throwing an eccentricity into the end joint which must be taken care of in some way, either by moving the reaction as shown, or strengthening the bottom chord at that point. No set rules can be laid down, but in any case the stresses in the joints should be figured carefully and the cuts made adequate.

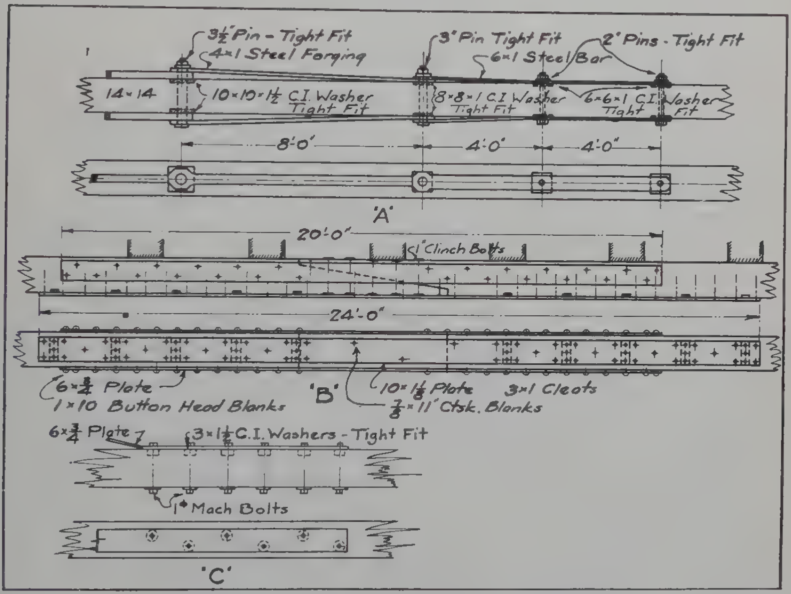

FIG. 9. — SPECIAL JOINTS AND SPLICES

In Fig. 9 detail "A" shows a method of bringing a heavy load into a tension member. Assuming a load of 240,000 lbs. one-half coming in on each side of the timber by means of eyebars, the loads will then be carried into the timber, as follows:

Washer 10 ×1½ = 15 sq. in.
Pin 3½×5 = 17.5 sq. in.
32.5 sq. in. @ 1,600 lbs. per sq. in. = 52,000 lbs.

Washer 8×1 = 8 sq. in.
Pin 3×4 = 12 sq. in.
20 sq. in. @ 1,600 lbs. per sq. in. = 32,000 lbs.

Washer 6×1 = 6 sq. in.
Pin 2×3 = 6 sq. in.
12 sq. in. @ 1,600 lbs. per sq. in. = 19,200 lbs.

Washer 6×1 = 6 sq. in.
Pin 2×3 = 6 sq. in.
12 sq. in. @ 1,600 lbs. per sq. in = 19,200 lbs.
122,400 lbs. each side.

The washers are simply square cast-iron plates with pin holes, and are fitted tightly into the timber, care being taken to match the pin holes with those in the steel side bars, so that all parts will be uniformly stressed. The above figures are based on the actual bearing of the washers and on a bearing of the pins for about 1.5 diameters, or preferably a little less.

Detail "B" shows a tension splice as designed for certain peculiar conditions, which is a combination of several different methods. First the timber has a scarf joint, then it has on the bottom a steel plate with cleats, and on the two sides steel plates fastened with button-head blanks. As shown there are cross timbers on the top of the log which made impossible any fastening on that side. The joint as shown is figured for 200,000 lbs. The button-head blanks are used on the side plates to avoid the necessity of attempting to bore through from one side to the other and meet the holes in the second plate fair. By slightly moving the plates so they are not exactly opposite each other the bolts do not foul, and by using 1/16″ drift there is no danger of their pulling loose under load.

Detail "C" is somewhat similar to that shown in "A" except that it is more suitable for lighter loads. This involves the use of round cast-iron washers. The holes are scribed from the side plate, centered, counterbored for the washer, and then bored for the bolt. In this way it is possible to secure good bearing on the washers with a minimum amount of work. In this case no bearing is figured on the bolt but only on the washer, using the full bearing stress on the projected area of the washer. The detail shown has a value of 43,200 lbs.

In Fig. 10 are shown a few questionable methods in mill building construction and others which are superior. "A" shows a common construction with the column cut and the girders running through. This, of course, produces excessive stresses in compression perpendicular to the grain. "B" is preferable, but is still open to the same objection, even though a hardwood cap is used. If, of course, a metal cap is used that eliminates the trouble. "C" is preferable to "A," but is placing considerable dependence on the bearing of the bolts and the side grain compression against the notch. "D" gives a very substantial construction in an "all wood" detail.

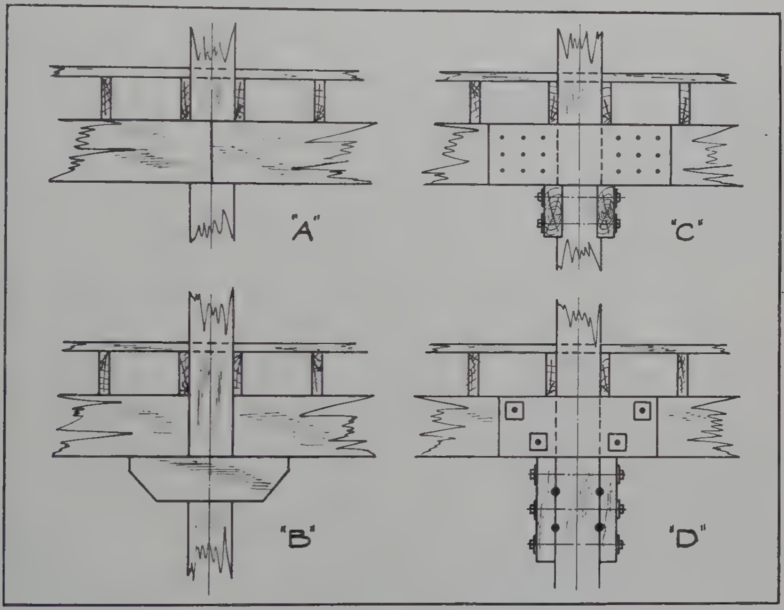

FIG. 10. — MILL BUILDING CONSTRUCTION DETAILS

Showing certain common methods together with improved designs

Before closing I should like to mention one detail of pier design which is in common use in Boston, namely, the double girder cap. Most piers constructed here use this double girder cap on the piles to support the deck system, whereas in other ports the single cap seems to be preferred. When these two types are considered from the standpoint of strength and economy the single cap appears far superior. It affords a much greater bearing surface between the cap and the pile, and eliminates any necessity of relying on the bolting to carry any part of the load. It also does away with the slow costly fitting of the pile heads, making for much speedier construction with the subsequent saving in time and money.

This covers in a general way the details usually encountered in timber design. The methods shown, either alone or in combination, will ordinarily be sufficient to solve the majority of cases. Although there is a growing tendency to neglect timber, it is still an important structural material, and, when properly handled, with due consideration for its peculiar characteristics and limitations, it will be found to have a definite economic value, and in many cases to be superior. For this reason the engineer should familiarize himself with these characteristics so that he may be better fitted to make use of this material as occasion arises.

CHLORINATION OF WATER AND SEWAGE

Discussion*

By Linn H. Enslow

Linn H. Enslow (by letter)†: That the writer should have been requested to offer discussion of *Professor Phelps*' paper on "Chlorination of Water and Sewage" is indeed considered as a rare honor and privilege. The discussion is written with the idea that recent experiences lead to the belief that some slight additional information may be presented, and, further, that certain chlorination data obtained over the past year strengthen the predictions and early observations of *Professor Phelps*, who is now considered as the pioneer in this country in the matter of chlorination of sewage.

If reference is had to the chapter "Chlorination" in the "Manual of Water Works Practice" recently published by the American Water Works Association, which chapter was prepared by the writer, it will be found that attention is directed to the following statement made by *Professor Phelps* as a part of expert testimony given during the now famous Jersey City case in which the "pros and cons" of water chlorination were brought out. During hearing of evidence in this trial in 1908, *Professor Phelps* disagreed with other experts who explained that the efficacy of chlorination was attributed to the liberation of nascent oxygen through the action of chlorine on the water molecule, and that it was this oxygen that caused the sterilization. *Professor Phelps* stated, "The chlorin in itself is toxic to micro-organisms."

Mainly because substantiating evidence at the hearing could not be presented, the theory of *Phelps* was not accepted. Since then, however, data from many points have proven the correctness of *Phelps*' assumption. The first and most conclusive data showing the tenability of the toxicity theory were presented by *Race* of Ottawa in 1916, and appear in his book on "Chlorination."

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* Discussion of paper by Prof. Earle B. Phelps, published in the April, 1926, Journal of the Boston Society of Civil Engineers; presented before the Sanitary Section of the Society on February 3, 1926.

† Sanitary Engineer, The Chlorine Institute, 30 East 42d Street, New York, N. Y.

In a statement made by the writer in 1916, attention was called to the fact that chlorine should be more effective, or rather more quickly effective, in the sunlight than in the dark. This prediction was based upon the fact that sunlight hastens the liberation of oxygen from water. If this were true, it was wondered if the low sterilization efficiency at times was not due to the fact that no light reached the chlorinated water after application of the chlorine. An investigation and study revealed the fact that the samples exposed to sunlight showed lower sterilization efficiency than did those kept in the dark. The difference was slight, but was of sufficient interest to warrant further study. The final outgrowth of the investigation was the improvement of chlorine control and the introduction of the residual free chlorine method. The necessity of providing sufficient chlorine to satisfy the demand by the organic matter, and a slight but distinct excess of residual in order to insure satisfactory bacteria removal, is but another bit of confirmative evidence as to the correctness of *Phelps*' theory or assumption of 1908 regarding the true rôle of chlorine in water sterilization in the face of the universally accepted "nascent oxygen theory."

Professor Phelps warns against the dependence upon the ortho-tolidin control tests for regulation of chlorine dosage without the checking of the efficacy by bacterial analyses. This is, of course, an important matter and should be carried out, but there are many minutes in the day when a sample taken for bacterial analysis could not indicate the efficiency of treatment until twenty-four to forty-eight hours later. In consideration of this fact, it is apparent that the ortho-tolidin test, which is easily and quickly made, has advantages which the bacterial test does not. It can be made with as great frequency as is thought essential and with but little cost or effort. It can also be made by the *untrained* personnel. Considering its many advantages, its adoption and use provide the small plants with a control which is little, if any, inferior to that which might be given by a resident bacteriologist. The adoption of this control has made group supervision of many plants by one chemist or bacteriologist feasible, and with very satisfactory results. Formerly, the quality of finished water could be determined infrequently for such plants, and the dosage of chemicals set at one visit was not changed until another, notwithstanding the fact that the pre-chlorinated plant output was subject to changes in quality.

That there is merit in the ortho-tolidin control test can be attested by the sponsoring of this method of control of chlorination by the various state departments of health and the provincial health departments in Canada. With the exception of the instance reported showing the difficulty of proper sterilization of an army water supply when regulating the dosage by the residual test, no other report of such nature has come to the writer's attention. An exception must be made, of course, to grossly polluted waters not having had preliminary purification of some kind. Even such waters may be rendered sterile through maintenance of somewhat higher residual chlorine than is commonly recommended for relatively clear and clean waters. *Professor Phelps* correctly advises against the assumption that a *given* concentration of residual chlorine will insure equal efficiency of bacteria removal from a sewage effluent or polluted supply containing considerable colloidal or suspended matter as would apply to a clean water provided with a like concentration of residual chlorine.

At this juncture it is desired to call attention to a study recently made by the State Department of Health of New York, the State Department of Health of Connecticut, and later by others at Dallas and Austin, Tex., under the supervision of the Texas Health Department. It appears from the results that in the chlorination of sewage and sewage effluents, the efficiency of the process can be fairly well determined by the presence or the absence of residual chlorine as determined by the ortho-tolidin test. Until residual chlorine is indicated, the bacteria removal efficiency is low. Even more so than in the case of waters, different sewages or effluents possess a varying demand for chlorine by the organic matter *in solution*, and therefore the control is subject to variations. It is believed that this is not alone the case for various sewage effluents, but is more than likely true for the same plant effluent at various seasons during the year. There has already been gathered data which tend to show a decided variation in chlorine demand by a given sewage between summer and winter conditions. As brought out by *Professor Phelps*, there is a marked difference in chlorine required for sewages of different degrees of septicity. The rule seems to be: the fresher the sewage — the less the chlorine required for effective results. This is true most especially in sewage which is carried in water initially high in sulphates. The sulphates are readily changed to sulphides and hydrogen sulphide. The latter requires two pounds of chlorine for each pound present.

In discussing the effect of the presence of hydrogen sulphide on chlorine consumption, it is interesting to note that *Cohn* at Schenectady, N. Y., has conducted extensive studies covering the application of chlorine (both chlorinated lime and liquid chlorine) to the Imhoff tank effluents prior to application to the sprinkling filters. The result was an immediate reduction of odors surrounding the plant. Other interesting results followed, also, and as *Cohn* expects to publish them, it is desirable to merely mention the odor elimination in connection with the statements covering the effect of hydrogen sulphide on chlorine consumption. Here, again, the rôle of the chlorine is not an oxidizing action. The chlorine enters into direct combination with the hydrogen and the sulphur is precipitated as atomic sulphur in a very fine state of division. The precipitated sulphur can, however, be settled out and for certain purposes possesses a value. Sulphur in this form is readily available as a fungicide spray for plants or trees. It should also be very satisfactory for making lime-sulphur spray. *Dr. H. Bach*, Chief Chemist of the Emscher Corporation in Essen, Germany, in his interesting paper published the latter part of 1925, reports findings and conclusions which in every respect corroborate and check the findings of *Cohn*. In view of the fact that the two experiments were being conducted here and in Germany at the same time, it is interesting to learn that the results are in agreement. Both *Cohn* and *Bach* find that small quantities of free residual chlorine applied to the biological beds have no detrimental effect upon the proper biological functioning of these beds. *Bach* also stresses a statement which proves to be a fact in practice: "The rate at which chlorine is absorbed when applied to a sewage is dependent, other things being equal, upon the state of subdivision of the absorbing matter." Gases will absorb quickest; solids in true state of solution next; third, solids in semi-state of solution or commonly considered as in the colloidal state. Finally, the true solids in suspension will be attacked but not until the demand for chlorine by the preceding has been satisfied. The absorption by sewage solids is relatively slow. Once a residual chlorine dosage has been provided which will remain at the end of a fifteen-minute period of contact, it is interesting to observe the number of hours this residual will remain in fresh sewages unless the solids are sufficiently agitated to be broken up, and so expose constantly fresh surfaces to the liquor containing the free chlorine. It appears that once the chlorine demand of the liquid phase has been met and the surface of the solids has absorbed a quantity sufficient to produce a balance for the given concentration, no further rapid absorption or adsorption takes place until additional chlorine is added to raise the concentration of the free chlorine in the surrounding liquid.

A further recent observation made during the chlorination of a very strong effluent from an Imhoff tank is the effect of increasing or decreasing the pH value prior to chlorination. The chlorine demand is found to increase in both directions as the neutral value pH 7 is departed from. The exact cause for this has not been determined. The tendency, however, will be toward the formation of chloramines and hypochloritess a the pH rises above 7, and as the pH recedes below 7 the tendency is toward the formation of nitrogen trichloride. It is further interesting to note that observations disclose that chlorine causes a reduction of the biochemical oxygen demand of the treated effluent. This reduction begins with small quantities of chlorine added and increases with the increase in chlorine applied. Such decreases are noted before sufficient chlorine has been added to render even satisfactory bacteria reduction and well before there is a residual of free chlorine found. In determining the B. O. D. of the chlorinated sewage in every instance the sample was reseeded with unchlorinated sewage after the dilutions had been made in the incubation bottles. Where the chlorinated sewage contained residual chlorine, this was neutralized immediately upon taking the sample; the dilutions were then made within forty-five minutes, and these were also seeded with fresh sewage bacteria.

Sewage effluent which was partially charged with dissolved oxygen by aerating, when treated with even small quantities of chlorine, *i.e.*, about one-third of the chlorine demand value, showed some dissolved oxygen present after twenty-four hours standing at room temperature. The non-chlorinated sewage had long since become septic and very odorous. This sewage contained considerable packing-house wastes and was for this reason very unstable. With higher dosages the residual dissolved oxygen increased. This indicates a reduction in rate of putrefaction, and, taken with the actual reduction in ultimate oxygen demand as determined by the five-day B. O. D. test, the indications are that not only is putrefaction retarded, but it is actually *reduced*. It is further reasonable to assume, although no twenty-four-hour B. O. D. determinations were made, that the largest part of the reduction of ultimate oxygen demand must have been brought about through a combination of the chlorine with the matters in the finest state of subdivision and therefore the most absorbent compounds. If this is, as seems, worthy of assumption, then the immediate oxygen demand is the first reduced by chlorine, and also is that part which is the cause of immediate nuisances in small streams which are burdened with the sewage until a larger body of dilution water is reached. A reduction, then, of the immediate nuisance, and at the same time the ultimate or total oxygen demand of the sewage in addition to the removal of bacteria, indicates for chlorine a value in addition to that usually credited to it in sewage disinfection practice. The studies still in process will be reported upon at some future time, at which the numerical data and curves will be presented in substantiation of the above statements.

Returning now to further discussion of the chlorination of water: the writer cannot help referring to *Professor Phelps*' example of failure of chlorination as that of Charleston, W. Va., in 1917. With such equipment as the filter plant had, there is some question as to whether it could have been made to produce a safe — if not clear and sparkling — effluent. With all of the variables, such as overload on the filters, frequent changes in rate of pumpage, and the manually controlled chlorinator, the chances for the 1917 outbreak of typhoid in Charleston would have been reduced materially, if not eliminated, had the plant been at that time operating on an ortho-tolidin control basis. Just such an occurrence is one of the best and most convincing arguments that can be presented to show the merits of the ortho-tolidin control in emergencies. Had there been a competent bacteriologist in charge of this plant, the writer doubts if he would have, during such a time as described, traded his bottle of ortho-tolidin solution for petri dishes and test tubes with the necessary media to put in them. This is only stated to show why the writer defends so strongly the true value and merits of the residual-chlorine control. It merely resolves itself into a matter of deciding whether it is only essential to maintain 0.1 p. p. m., 0.2 p. p. m., and, in cases similar to the Charleston case, 0.3 p. p. m. or more after a contact period of ten minutes. As to the matter of contact period, this feature becomes more important as the turbidity or colloid content of the water increases. The writer has just seen a set of curves indicating that the contact period is relatively unimportant when the optimum chlorine dosage has been applied to a relatively clean water. The bacterial reduction at the end of five minutes was as complete as at the end of an hour. Where the water is underdosed, and the bacterial reduction is dependent upon the chloro-substitution products formed, the contact period becomes of added value. This is true because of the relatively slow action obtained from these somewhat toxic compounds formed. The more *dependable* procedure, then, is to provide sufficient chlorine to obtain the residual test to the extent desired and insure a rapid removal of organisms.

If we can judge by the results secured by *Mr. Bahlman*, who is in charge of the Cincinnati filter plant, which practically indicate with certainty that a definite improvement in bacterial quality of the Cincinnati tap samples could be attributed to the introduction of chlorine control by the ortho-tolidin method, dependence upon this method of control rather than upon the bacterial quality of the plant output without residual chlorine control (as has been the general practice in the past) will lead to general improvements in quality of chlorinated waters which heretofore have not been obtained with the bacterial control methods alone. Those cities or water companies maintaining that it is not possible, within practicable limits, to produce a finished water at the services which will pass the proposed United States Treasury Department Standards for water served on Interstate carriers, will likely find it advantageous to adopt the methods of *Mr. Bahlman*, thus reducing the probability of "aftergrowths," and, more particularly, eliminating chance contamination from unlooked-for disturbances throughout the storage reservoirs and distribution system.

The writer can cite an interesting case within his knowledge which touches on the disturbance factor in mains and storage basins holding purified water. The filtration plant, placed in operation a year previous to the study made, had been producing continuously a satisfactory finished water. This water was stored in a reservoir which for years previous had been receiving non-filtered water. The bottom and sloping sides were covered with slime and deposits of varying thickness. As was the case in the past, early in the summer the B. coli content of the tap samples suddenly increased without explanation which could be had from a review of the results of the samples examined as the water left the plant, and likewise from the records of samples taken from service taps on the high-service system which was supplied from a covered tank. Investigation of the matter disclosed the fact that the slime on the bottom of the old brick-lined reservoir harbored a bacillus which, in accordance with the Standard Methods for Water Examination of the American Public Health Association, could not be classified other than true *fecal* coli. This was confirmed by several laboratories to which cultures of the organism were sent; among these was the laboratory of the United States Public Health Service. In each case, directly above the point at which the slime sample was collected from the sloping bottom, a sample of the supernatant water was collected at a point about 3 feet below the surface. In none of these water samples could the colon bacillus be detected and the total agar count was always within the range of 5 to 50 per c.c. Other similar experiences have been met where an organism classified as the colon bacillus apparently lived in deposits and coatings within the pipe mains. To demonstrate this, samples from taps on these mains had consistently shown highly satisfactory results. To show the existence of living B. coli in the deposits or film, it was only necessary to open fire plugs, allow them to waste a while, and thereafter collect samples from the same service taps as had for some months previous, and also the same day, shown excellent water bacteriologically. B. coli could repeatedly be found in the samples, although to a limited extent. These interesting bacteria, classified as fecal B. coli, would grow in citrate broth, and therefore, according to *Koser*, could be classified with the several strains of the colon group which have been isolated from certain soils which could not in any manner have been contaminated by man or beast.

The above has been brought to attention primarily to show that when trouble arises, considerable can sometimes be explained through a survey, and often so-called "aftergrowth" may prove to be something else. The presence of a slight quantity of free residual chlorine in addition to prevention of "aftergrowth" will likewise take care of minor disturbances within the reservoirs or mains, and also clear up contamination that accidentally enters at points where connections, replacements and repairs are made.

Professor Phelps has mentioned a point covering the method of chlorine application at two or more points rather than dependence upon a single application. Up to the present this has not been widely recognized nor appreciated except at a limited number of places, among which should be mentioned, in addition to the city of New York, the following: Albany, N. Y., has found it advantageous to chlorinate the raw Hudson River water prior to coagulation. This is not for the purpose of reducing the coagulant used, but rather is a means of safeguarding further the supply through a preliminary reduction of the bacterial load which otherwise would have to be handled by the filters. The filter effluent is later chlorinated at two additional points with ortho-tolidin control of dosage. Naturally, the water already having been pre-chlorinated and filtered does not require the same chlorine to provide a residual in the plant output, as would be the case if no prechlorination had been given. Under *Mr. James W. Caird*, Consulting Bacteriologist of Troy, N. Y., there are a number of large as well as small plants, over which his firm has operation supervision, where, if there is an apparent risk which might result as an outcome of lapse of efficiency of the filters, the raw water is continuously pre-chlorinated with a fixed rate of application in order that the raw water bacterial index peaks may be flattened. As a result, the filter loading becomes more or less a constant. At Toronto, pre-chlorination in addition to chlorination of the final effluent is also practiced. The final chlorination is under residual chlorine control. At Dallas, Tex., the filter effluent is chlorinated as it enters the open filtered water basin. As it leaves the plant and enters the mains it receives a second dosage which, though light, is sufficient to provide 0.2 p. p. m. residual. The results have been very satisfactory and the method has proven effective in reducing the frequency with which samples from services have unduly high counts. The practice of split dosage or dual chlorination will come to be the more appreciated as the load on the plant increases, whether it be rate of filtration or quality of the pre-filtered water.

The writer has reserved for the closing portion of the discussion that very pertinent matter of tastes produced in chlorinated water through a combination of the chlorine with organic compounds within the water or the pipe lines. The odor produced by excess chlorine is not worthy of discussion, as it is soon recognized what residual chlorine the particular water will stand and not possess at the services a true chlorinous odor. In general the higher the soluble organic content, the greater the free chlorine which can be carried without the production of true chlorinous odor, and, conversely, the higher the *free* chlorine carried, the less the likelihood of producing or leaving a residual organic-chlorine by-product which will impart to the water disagreeable tastes. On the other hand, the less the organic content, the bicarbonate content, and more especially the free carbon dioxide content, the less can be the free chlorine content maintained without producing *chlorinous* taste or odors at the services, — particularly at hot-water services.

The rate of dissipation of the residual available chlorine is governed by the pH value of the water treated. In a softened water from which practically all, if not all, of the bicarbonate has been removed, for example, a good portion of the chlorine goes to form hypochlorites and probably also chloramines. These are very slow in activity and suffer loss of their available chlorine in the absence of free carbon dioxide or considerable bicarbonate. A softened water which has not been carbonated, or a water which has received lime and iron sulphate coagulation, will likely have such a scarcity of bicarbonates as to be indirectly responsible for the "medicated" or "iodoformish" tastes at the services. The assumption to account for this condition is as follows. The chlorine added produces a hypochlorite. The available chlorine persists within the mains because of the scarcity of hydrogen ions necessary to hasten liberation of free chlorine. At the softening or filter plant, the chlorinated water, although tasting flat and somewhat "mawkish," — to quote *Sir Alexander Houston*, — does not become really objectionable until reaching the older sections of the mains or dead ends. At the services the farthest away from the plant the available chlorine may be almost as great as when it leaves the plant. It will be found, however, that the maximum color from the ortho-tolidin reaction is not obtained spontaneously as was the case with tests made at the plant or close by. On the contrary, ten to fifteen minutes are required for the maximum color production. The writer has always found this slowness of color production to be associated with bad tasting water, and it is taken as an indication of what to expect later if the taste is not already apparent. It appears almost a certainty that the available chlorine in the alkaline medium attacks the pipe dip or paint on the standpipe or tank walls. If this dip or paint had a coal-tar base, as was frequently the case in years past, chlor-phenols are formed within the mains. This idea is further substantiated because of failure to produce the strongly iodoformish taste when the same water is held in storage in carboys at the plant.

At Dallas, Tex., a study was made which pointed to relief being obtained from the iodoform tastes by a change in coagulant control. If this had failed, it was proposed to carbonate the final effluent at the same point at which chlorine is applied. To date the carbonation has not been necessary. Careful control of the lime and iron dosages has resulted in a better tasting and appearing water, and at the same time one which contains twenty parts per million of bicarbonate and approximately fifteen to twenty parts of normal carbonate. The effluent is sparkling as a result of a slight increase of iron sulphate and a decrease in lime. The applied chlorine does not persist in the mains. The tastes have been almost entirely eliminated and the bacterial quality has continued satisfactory. To make the results yet more interesting, there has been an overall saving, due to reduction of chemicals. Somewhat less chlorine is required, and only a trace of residual can be found at the services in the business district of Dallas. There may appear later in the season a sufficient reason to add more lime in the coagulation, but should this be the case and no bicarbonate alkalinity, or but little, be had in the filtered water, carbonation will be adopted to carry over such periods of raw water condition as require the caustic adjustment and high pH for effective or economical coagulation and softening.

In cases of high pH and absence of bicarbonates increase in chlorine fails, for reasons cited, to remedy the tastes. In cases of tastes produced as a result of decayed organic matter or algæ, it appears feasible to increase the chlorine to a point where sufficient residual is had to provide a true chlorinous odor. Under such condition it appears that the by-product is destroyed by the excess-free chlorine which later disappears as the water passes through reservoirs and the mains. Super-chlorination is particularly effective as a preventive of "weedy" and "fishy" tastes. *Delaporte* in Ontario, *Hale* of New York, *Bushnell, Howard* at Toronto, the writer, and others have found the increased chlorine to be effective in producing improvement. Even should the dosage be sufficient to cause a slight *chlorinous odor* at the services, this condition is considered by the average consumers to be much less objectionable than the *medicated tastes*. Waters deficient in dissolved oxygen as well as those containing algæ or decaying organic matter are in particular amenable to the slight super-chlorination process. The first outstanding example of the success of this form of treatment in America is that reported by *Hale* of New York. A small dosage in combination with synura produced very offensive tastes and fishy odor in the Croton supply of New York City. Increasing the dosage so as to insure residual chlorine of more than 0.2 p. p. m. cleared up the trouble. The free chlorine in excess was entirely dissipated in passing through the reservoirs and flumes.

Professor Phelps has touched on a very important matter, viz., that of a thorough and rapid mixing of the chlorine and water. Both Chicago and New York City some time ago had experience with low efficiency of bacterial removal where chlorine was being applied in large flumes and pipe-lines. With no increase in the dosage applied, but merely introduction of baffles or bars to cause rapid mixing, the efficiency was materially increased. Even in applying chlorine directly into small mains much improvement in efficacy can be obtained if some valve, elbow or other impeding object is just beyond the point of application to produce rapid mixing.

The point was mentioned that the greater the concentration of chlorine applied, the greater and more rapid would be the absorption of the residual chlorine by the organic matter present. This is very fortunately the case, and is the principal reason why an organic water will stand overdosing with chlorine and the excess be forced into the compounds already formed during the early stages of chlorine-organic matter contact. In other words, the greater the overdosage, the greater is the tendency for increased absorption. The excess chlorine in the free state does not increase in proportion to the additional chlorine applied. This condition of affairs is a real asset in the matter of plant operation where taste reduction or prevention is sought.

Sir Alexander Houston of London, who has had very considerable experience in taste eradication, very aptly expresses the situation in the following: "At times of taste troubles it is usually '*chlorine-boldness*', rather than '*chlorine-temerity*,' which is needed in attempts to improve the situation."

BY E. SHERMAN CHASE

E. SHERMAN CHASE (by letter)*: The paper of *Professor Phelps* suggests many interesting topics for discussion. The writer, however, will confine himself to pointing out briefly the rapidity and ease with which emergency chlorination of public water supplies can be established.

As long ago as 1914 the State Health Department of New York, through the foresight of *Mr. Theodore Horton*, then its chief engineer, purchased a small chlorination apparatus for emergency installation on water supplies responsible for outbreaks of water-borne typhoid fever. Since that year the Department has had numerous occasions to utilize its chlorination equipment to meet serious emergencies of water supply pollution and infection.

During the war, for example, the city of Watervliet, N. Y., was obliged to supplement for a time its regular water supply by pumping directly into the mains from the Hudson River. Above the point of intake the Hudson or its tributaries had received the sewage from nearly half a million people, including that from the city of Watervliet itself. The State Health Department installed its emergency chlorination apparatus prior to the pumping of any river water. By heavily chlorinating this water it was possible to prevent an outbreak of typhoid which past experience had shown would have inevitably occurred.

Previous to the availability of portable and easily installed chlorination equipment, sanitarians during outbreaks of water-borne typhoid had to content themselves with futile warnings regarding the need for the boiling of drinking water. Now, when polluted supplies must be used to meet some unusual or unexpected condition, chlorination apparatus can be quickly installed, and, with proper operation, no serious consequences need result.

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* Sanitary Engineer, with Metcalf & Eddy, Consulting Engineers, Boston, Mass.

OF GENERAL INTEREST

ALDEN HYDRAULIC LABORATORY

Prominent Hydraulic Engineers Attend Opening of Enlarged Hydraulic Laboratory of Worcester Polytechnic Institute on May 7

Through the generosity of George I. Alden, formerly Head of the Mechanical Engineering Department of Worcester Polytechnic Institute, the Alden Hydraulic Laboratory has been materially enlarged and improved during the past two years. On the occasion of the opening of the new building on May 7, 1926, the entire plant was placed in full operation, illustrating various methods of water measurement; dam, spillway, and intake model testing and many other hydraulic experiments.

The guests, numbering over a hundred, included prominent hydraulic engineers, professors of hydraulic engineering in technical schools, and manufacturers of hydraulic equipment. In place of formal exercises, the entire time, except for luncheon, was devoted to inspection of the plant in operation, group discussions, and the meeting of men interested in hydraulic engineering.

The Alden Hydraulic Laboratory is located in the village of Chaffins, in the township of Holden, five miles north of Worcester Polytechnic Institute. This site, formerly occupied by a woolen mill, was discovered in 1894 by Prof. George I. Alden, then Head of the Mechanical Engineering Department of the Institute. Hon. Stephen Salisbury, President of the Board of Trustees, presented it to the Institute. Mr. Charles H. Morgan had the plans drawn and obtained the first piece of apparatus installed, — the 36-inch Venturi meter which had been at the World's Fair in Chicago in 1893. In 1910 the original low-head laboratory was built, and in 1925 the new main laboratory building was completed, both through the generosity of Mr. Alden.

The Institute owns the flowage rights on three ponds, — the upper one, No. 1, of two hundred acres used for storage; No. 2, of four acres, discharging through 400 feet of 40-inch riveted steel pipe to the main laboratory under about a 30-foot head; and No. 3, which supplies water to the low-head laboratory under a head of about 15 feet. The land owned by the Institute consists of one hundred acres extending downstream to the land of the Metropolitan District.

One of the chief assets of the laboratory is the complete control of the water from the storage reservoir down to the Metropolitan District. The flow can be stopped entirely for repairs or new construction, and varied from minimum to maximum without damage to any one on the river below, as all of the water eventually flows into the Wachusett Reservoir.

The main building is wooden of mill type construction, 110 feet long, 45 feet wide and 15 feet high, with two towers running up above the pond level, one for head-measuring water columns and the other for the surge tank. Other buildings include the low-head laboratory, office building, house over stream gaging apparatus and storehouses.

Although space does not permit of a complete description here, it may be said that the equipment is extensive and permits of carrying out experiments of all kinds in hydraulic engineering, such as flow measurements, current meter ratings, water wheel and pump tests, investigations of spillway and draft-tube types, etc.

In addition to its use by the students of Worcester Polytechnic Institute, Tufts College has for a number of years sent its engineering students to this laboratory for their hydraulic experimental work. With its improved facilities the Institute will now be able to increase its service to the hydraulic engineering profession. Charles M. Allen, Professor of Hydraulic Engineering at the Worcester Polytechnic Institute, who is in charge of the Alden Hydraulic Laboratory, has been connected with this work for a number of years, and is in large measure responsible for the successful development of this project.

Moment Curves for Moving Concentrated Loads

By Waldo F. Pike, Structural Engineer, Kendall Square, Cambridge, Mass.

Several years ago, when working in a railroad office, the writer came across a method of plotting moment diagrams for moving concentrated loads, which apparently is not generally known. It involves the computations for and plotting of the critical points of the diagram only.

A general description would be lengthy and perhaps difficult to visualize. A concrete example will bring out the method which it will be seen is general in its application to any group of moving concentrated loads.

The two-car train specified in the "Specifications for Bridges Carrying Electric Railways" of the Massachusetts Public Service Commission of 1915 will be plotted. This example is chosen because of its simplicity and its usefulness to those engaged in bridge design.

For Loading Diagram, see Fig. 1.

First, *one wheel* on the span will be considered. The minimum span for one wheel on is zero. The maximum span for only one wheel on, with the wheel in the center of the span, is 10 feet. The moment for one wheel on a 10 ft. span is —

$$12{,}500 \times \frac{10}{4} = 31{,}250 \text{ ft. lbs.}$$

This moment is plotted and a straight line is drawn from zero to this point indicating a maximum condition for one wheel on the span.

Next, *two wheels* on the span will be considered. The absolute maximum moment for two wheels on, comes when the center line of the span lies halfway between the resultant of the wheel loads and one of the wheels. The minimum span that will contain two wheels with the above condition satisfied is 7.5 feet. The maximum span that will contain only two wheels and satisfy the condition for absolute maximum moment is 32.5 feet. The moment for the minimum span is —

$$12{,}500 \times \frac{5}{7.5} \times 2.5 = 20{,}800 \text{ ft. lbs.}$$

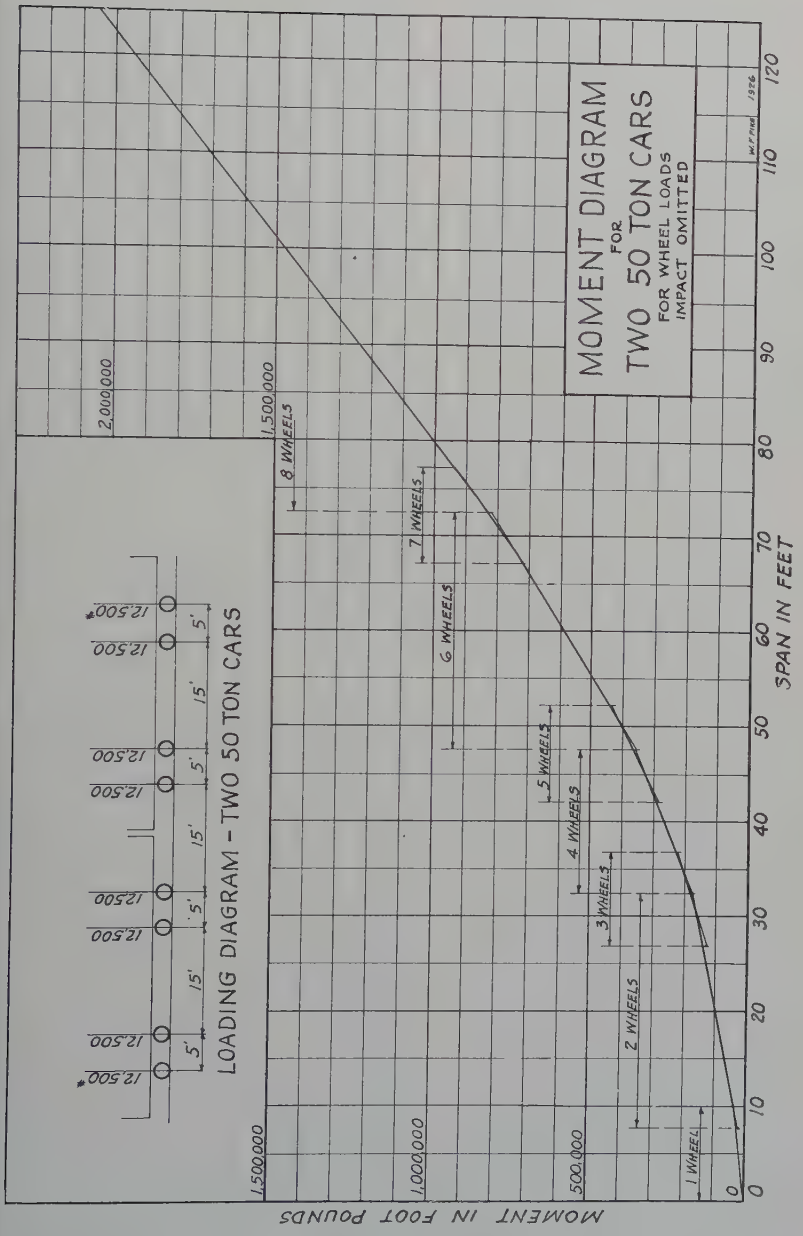

Fig. 1.—Loading and Moment Diagrams

The moment for the maximum span is —

$$25{,}000 \times \frac{15^2}{32.5} = 173{,}000 \text{ ft. lbs.}$$

These two points are plotted and a straight line drawn between them giving the absolute maximum moment for two wheels on the span.

It will be noticed that the moment for the minimum span, with two wheels on is less than the maximum with one wheel on, and that the moment lines cross in the interval between the maximum span for one wheel and the minimum span for two wheels.

Next, *three wheels* on the span will be considered. The maximum and minimum spans for this condition are shown in Fig. 2. The moment for the minimum span is —

$$25{,}000 \times \frac{9.17}{26.67} \times 15 = 129{,}000 \text{ ft. lbs.}$$

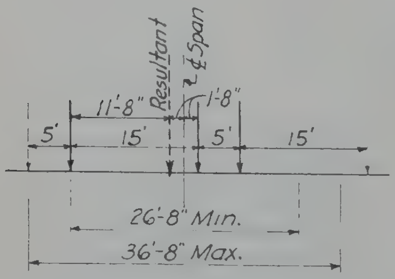

Fig. 2.— Maximum and Minimum Spans for Absolute Maximum Moment— Case of Three Wheels on

The moment for the maximum span is —

$$37{,}500 \times \frac{16.67^2}{36.67} - 12{,}500 \times 5 = 221{,}500 \text{ ft. lbs.}$$

These two points are plotted and a straight line drawn between them. This procedure is continued successively with four, five, etc., up to eight wheels. For eight wheels some arbitrary span convenient for plotting is taken for a maximum.

When all the points are plotted and the lines drawn connecting the minimum and maximum of each group of wheels, the plot will consist of a series of straight lines overlapping near the ends. To get the final diagram for maximum moment for the train use the upper lines.

Fig. 1 shows the Moment Diagram for the case described. The overlapping ends are left in simply to illustrate the description and should not appear in the final plot.

Arch Dam Investigation

Bulletin No. 3 of the Engineering Foundation contains a progress report on the "Stevenson Creek Test Dam." Bulletins Nos. 1 and 2, now out of print, directed attention to the importance of arch dams, and to the fact that their design has been based on theory unsupported by experiment, and outlined the proposed investigation and its aims.

The Stevenson Creek Test Dam is situated on a tributary of the San Joaquin River, about 60 miles east of Fresno, California. It is being built of Portland cement concrete, 60 feet high, with a vertical upstream face having a constant radius of 100 feet. Above the dam the gorge is so steep and narrow that the reservoir is small, and nearness to a main water-supply conduit of the Southern California Edison Company permits quick filling of the reservoir whenever required for tests.

When the concrete in the top of the dam is about three months old, water will be let into the reservoir and tests under pressure of water at various depths will be begun. Most of the tests will be repeated, and tests with water in the reservoir will be alternated with tests with the reservoir empty. Deflections, strains and temperatures will be measured under a variety of conditions throughout the year.

Should the dam built to a height of 60 feet continue to stand with the reservoir filled with water to its top, the dam may be extended in height, if the further tests which would then be made possible appear worth while and funds are provided.

Expenses for the test dam are being met by a special fund contributed by a number of interested companies and individuals, together with funds appropriated by Engineering Foundation. The total fund to date is $84,000, and $25,000 more will be needed to complete the program for the 60-foot dam. The dam would already have been completed to the 60-foot height except for serious floods in February, which caused unavoidable delay and increased expense.

These investigations are being carried on by a committee of members of the American Society of Civil Engineers under the auspices of Engineering Foundation. Besides the experimental dam, the committee is making tests on dams in service or under construction, and collecting information about many arch dams in the United States and in foreign countries.

When one considers the millions of dollars being spent in the construction of dams for power development, flood control, irrigation and water supply, the importance of securing data concerning dams, particularly of the arch type, is realized.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Designers Section

April 14, 1926. — The regular April meeting of the Designers Section of the Boston Society of Civil Engineers was called to order at 6.05 p.m. in the Affiliation Rooms.

The minutes of the March meeting were read and approved. Due to tne resignation of Mr. D. V. C. Birrell, a vacancy was made on the executive committee. The chairman asked for nominations for this office. Mr. E. D. Mortenson was nominated. A motion was made and carried that nominations be closed. A motion was made and carried that the Clerk cast one ballot electing Mr. Mortenson. The Clerk cast the required ballot.

The chairman then introduced the speaker of the evening, Mr. Ray H. Lindgren, Designing Engineer for the Crandall Engineering Company, who spoke on "Design and Detail in Timber Structures."

There were 50 members and visitors present. The meeting was adjourned at 7.30 p.m.

Scott Keith, *Clerk.*

APPLICATIONS FOR MEMBERSHIP

[May 15, 1926]

The By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of twenty (20) days from the date given.

For Admission

Baird, Charles Oscar, Lynn, Mass. (Age 25, b. Lynn, Mass.) Graduate of Northeastern University in 1922. Taught at Western Junior High School, Lynn, 1921–22; 1922 to date instructor in civil engineering at Northeastern University. Refers to H. B. Alvord, C. S. Ell, J. W. Ingalls, W. E. Nightingale.

Butterfield, William Rowe, Somerville, Mass. (Age 56, b. Bradley, Me.) Graduate of University of Maine. 1893–94 with French & Bryant on survey of Metropolitan Parks; 1895–98 in Public Works Department, city of Melrose; 1899–1902, survey and construction of Boston Elevated; 1902–04, assistant roadmaster, Boston Elevated; 1905, asst. roadmaster, Old Colony Street Railway; 1906–08, manufacturing automobile specialty; 1909, with Westinghouse, Church, Kerr Company; 1910 to date with Crandall Engineering Company as construction engineer. Refers to H. S. Boardman, J. L. Crandall, J. S. Crandall, C. T. Fernald, C. H. Gannett, H. C. Hartwell.

Cahill, Joseph E., Boston, Mass. (Age 54, b. Boston, Mass.) Educated in the Brookline schools and entered the building business with his father, taking up architecture in his uncle's office until he entered the employ of the city of Boston in 1898 as foreman of construction; transferred to the water department under civil service; later transferred again to the building department as supervisor, and is still in this department as chief of Plan Division. Has studied engineering under competent tutors and also taken a course in real estate law under the State University Extension Board. Refers to L. J. Monahan, M. D. O'Farrell, T. F. Sullivan, W. J. Sullivan.

Langstroth, Guy S., Boston, Mass. (Age 28, b. French Village, N. B.) Studied three years at Northeastern University; graduated from Wentworth Institute, School of Design. Experience: six years' construction and surveying with Aberthaw Company; Mass. Highway Dept.; United Fruit Company, and Phœnix Utility Company of New York. At present field engineer and building superintendent with the Phœnix Utility Company. Refers to C. F. Allen, H. B. Alvord, W. A. Bryant, E. A. Varney.

Stephens, Robert B., Jr., West Medford, Mass. (Age 25, b. Dorchester, Mass.) Graduate of Tufts Engineering School, 1922, with C.E. degree. March, 1923, to July, 1924, with Gulf, Colorado & Santa Fé Railway, at Galveston and Temple, Tex.; July, 1924–March, 1926, in the Engineering Division, Mass. Dept. of Health; at present with Fay, Spofford & Thorndike. Refers to F. L. Flood, N. L. Hammond, H. E. Holmes, F. H. Kingsbury, A. D. Weston.

For Transfer from Junior

MacDonald, Edmund John, Newtonville, Mass. (Age 26, b. Cambridge, Mass.) Graduate of Mass. Inst. Tech., 1921. Experience: road building and general contracting, 1922; coal mining, 1923; general contracting, 1924 to date. Refers to J. B. Babcock, H. L. Bowman, C. B. Breed, L. E. Moore, Hale Sutherland. (*Elected a Junior, September 30, 1924.*)

Murdough, Edwin B., Boston, Mass. (Age 28, b. Dorchester, Mass.) Graduate of M. I. T., 1920. Experience: rodman July, 1920, to November, 1920; with Metcalf & Eddy from February, 1921, to November, 1923; with George H. Sampson from May, 1924, to September, 1924; with Employer's Liability Assurance Corporation from September, 1924, to date. Refers to J. B. Babcock, H. P. Eddy, Jr., G. C. Emerson, J. P. Wentworth. (*Elected a Junior, January 25, 1922.*)

Wilkins, Henry Munroe, Marblehead, Mass. (Age 26, b. Marblehead, Mass.) Graduate of Northeastern University in 1921. During the four years at college worked as rodman and transitman with Whitman & Howard; transitman with Aspinwall & Lincoln, 1919–20; assistant engineer in Engineering Dept. of Land Court in 1921; since graduation has been employed as assistant engineer with Mass. Dept. of Public Works, Div. of Highways; now assistant engineer on maintenance, Mass. Dept. of Public Works. Refers to A. B. Appleton, A. W. Dean, A. L. Ford, L. B. Hoyt, A. P. Rice. (*Elected a Junior, March 15, 1922.*)

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Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street West, Toronto, Canada.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, William Penn Hotel, Pittsburgh, Pa.

Louisiana Engineering Society, State Museum Building, Jackson Square, New Orleans, La.

Montana Society of Engineers, Butte, Mont.

Rochester Engineering Society, Carnegie Building, University of Rochester, Rochester, N.Y.

Technology Club of Syracuse, 1158 Onondaga Hotel, Syracuse, N. Y.

Vermont Society of Engineers, Northfield, Vt.

Western Society of Engineers, 1735 Monadnock Block, Chicago, Ill.

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